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STURBRIDGE WASTEWATER TREATMENT PLANT

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WASTELOAD ALLOCATION

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PUBLICATION #16,248-51-25-3-90-C.R.
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STURBRIDGE WASTEWATER TREATMENT PLANT

WASTELOAD ALLOCATION DOCUMENTATION

By

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DECEMBER 1989

TITLE: STURBRIDGE WASTEWATER TREATMENT PLANT - WASTELOAD ALLOCATION

DATE: DECEMBER 1989

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SUMMARY

The Sturbridge Wastewater Treatment Plant (WWTP) discharges to the Quinebaug River at mile 20.2 and a 7-day, 10-year low flow (7Q10) of 6.82 cfs. In 1986 the 0.505 MGD secondary plant exceeded its 80% design capacity flow for more than 90 days. The town prepared a facility plan (and capacity report) as required by the NPDES permit and in 1988 issued a sewer moratorium. The continual growth in the town of Sturbridge and their imposed sewer moratorium has led the town to request an increase of 0.25 to 0.5 MGD in flow capacity at their treatment plant.

To determine what effluent limits would be needed for an expanded 0.75 MGD or 1.0 MGD plant, the EPA wasteload allocation (WLA) computer model Qual2E was used. The model was created to predict the effect of the Sturbridge WWTP's discharge on the Quinebaug River water quality. Documentation for the WLA is included within this report.

The model was created and verified using the two water quality surveys conducted by DWPC-TSB during the fall of 1988 (August 31, 1988 and September 20, 1988).

Once the EPA Qual2E model was created, 7Q10 low flow conditions were entered. Based on the model's predictions for dissolved oxygen at low flow, the following effluent limits are recommended for the Sturbridge WWTP:

RECOMMENDED AVERAGE MONTHLY LIMITS

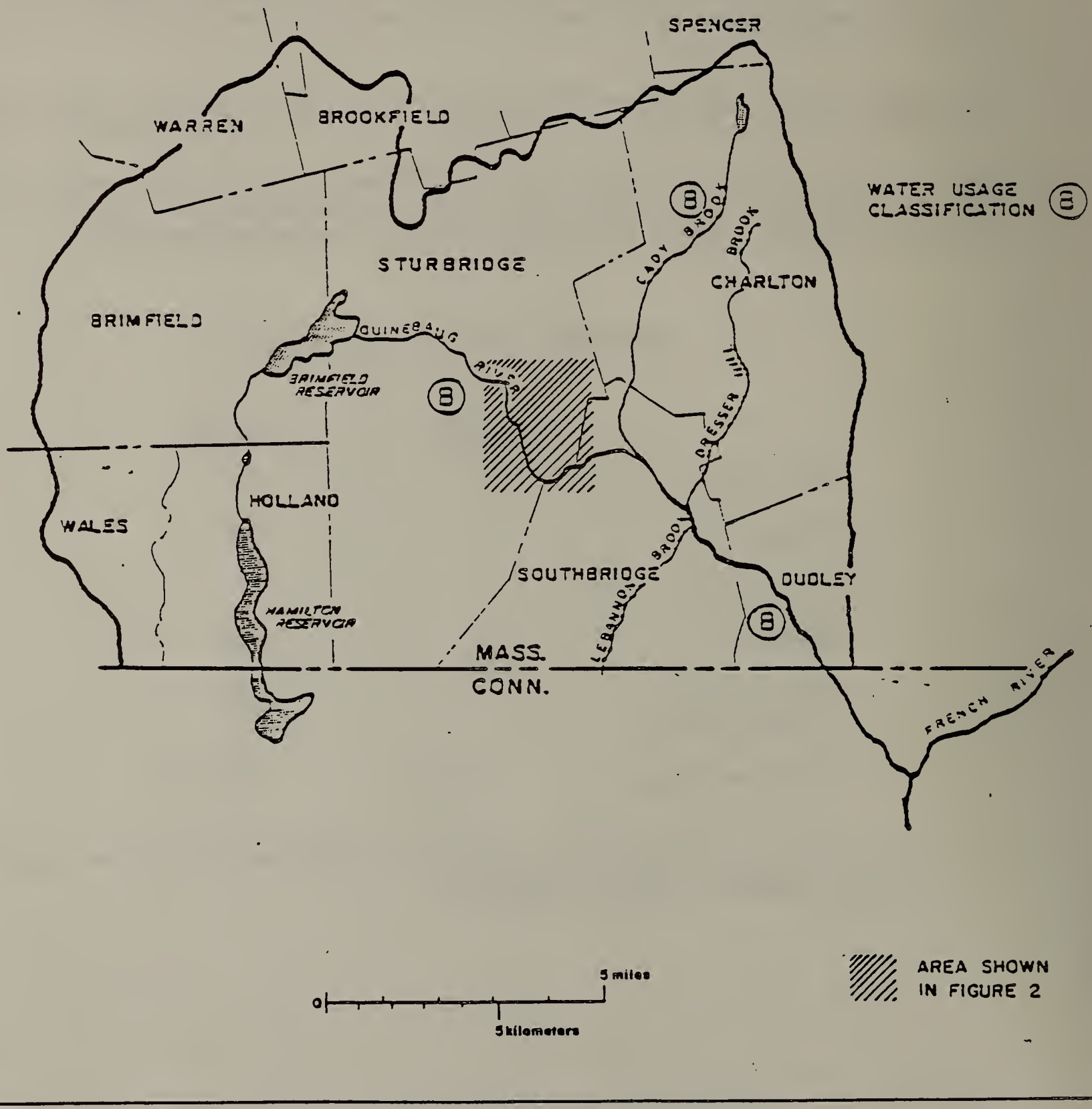
	0.75 MGD	1.0 MGD
BOD_5 (mg/l)	10	10
SS (mg/l)	10	10
NH_3-N (mg/l)	3	2
TP (mg/l)	1.0	1.0
Fecal Coliform* (org/100 ml)	200	200
Dissolved Oxygen	5.0 mg/l	5.0 mg/l

* If disinfection is done using chlorine, a limit will be set to protect instream aquatic life from toxicity.

Figure 1 QUINEBAUG RIVER BASIN

BASIN LOCATION

CLASSIFICATION MAP



DESCRIPTION OF QUINEBAUG RIVER BASIN

The Quinebaug River begins in Union, Connecticut and flows northwest to Hamilton Reservoir, which lies on the Connecticut-Massachusetts state line. Leaving the reservoir, the river then flows north through Holland and Brimfield in Massachusetts and turns east to Sturbridge, the river flows southeast to Southbridge, where it is joined by Cady Brook, and on through Dudley to recross the state line at Thompson, Connecticut. The Quinebaug River is then joined by the French River in Thompson, Connecticut and continues south to Norwich, Connecticut where it is joined by the Shetucket River; this confluence forms the Thames River, which continues south to Long Island Sound at New London, Connecticut. Of the Quinebaug River's 76 miles, 28 lie in Massachusetts and drain an area of 148 square miles. Municipal wastewater treatment plants in Sturbridge and Southbridge contribute to the flow in the Quinebaug River along with a cooling water discharges from the American Optical Company in Southbridge. Additional treated effluent is discharged to Cady Brook in Charlton by the town's wastewater treatment facility. (See Figure 1.)

The Sturbridge WWTP discharges to the Quinebaug River at mile 20.2. Below the Sturbridge WWTP the river flows through a narrow valley with occassional rapid sections till it becomes impounded by the Westville Army Corps Dam at river miles 15.7. The majority of this section is preserved open land to be flooded by the Army Corps during major rain or flood events.

INTRODUCTION

The Sturbridge Wastewater Treatment Plant (WWTP) Wasteload Allocation (WLA) computer model was created using water quality data collected on August 31, 1988 and verified with water quality data collected on September 20, 1988. The process for setting up the model, determining the reaction coefficients, verifying the data and running low flow data is contained in this report. A sensitivity analysis was run to determine the accuracy of the coefficients.

REACHES

The Quinebaug River from just above the Sturbridge Wastewater Treatment Plant in Sturbridge to Route 131 in Southbridge (0.8 miles below the Westville Dam) was modeled in the wasteload allocation. This 5.4 mile stretch was divided into six different reach segments. Segments were created to distinquish between rapid sections, straight laminar sections and the Westville impoundment. Each segment reach was divided into elements of 0.1 miles for the computers calculations. The six reaches are listed in Table 1.

TABLE 1

QUINEBAUG RIVER REACHES FOR QUAL2E MODEL

REACH	DESCRIPTION	RIVER MILES	NUMBER OF ELEMENTS
1	Sturbridge WWTP to Hobbs Brook	20.3 - 19.3	9
2	Hobbs Brook to Farquhar Road	19.3 - 18.9	5
3	Farquhar Road to Breakneck Brook	18.9 - 17.8	11
4	Breakneck Brook to inlet Westville Impoundment	17.8 - 16.7	11
5	Westville Impoundment - inlet to outlet	16.7 - 15.7	10
6	Outlet Westville Impoundment to Route 131, Southbridge	15.7 - 14.9	8

Figure 2

QUINEBAUG RIVER

LOCATION OF SAMPLING STATIONS

BASIN LOCATION

84

20

131

QRT01

QRT02

QRT03

QRT04

QRT05

QRT06

QR01

QR01A

QR01B

QR01C

QR01D

QR01E

QR01F

STURBRIDGE WWTP

QUINEBAUB RIVER

WESTVILLE RES.

Sturbridge

Southbridge

INSTREAM FLOW MEASUREMENTS

The Sturbridge model was created using data from the August 31, 1988 survey data and verified using the September 20, 1988 data. Instream flow measurements were taken from one stream location above the WWTP and the five tributaries which join the Quinebaug. (See Figure 2.) A stream flow measurement is also available at the Westville Dam. The flow measurements obtained during the surveys and input to the model are listed in Table 2. Both surveys had very similar instream flows. Ideally, the model would be calibrated and verified under two very different flow conditions. Since no other data were available, these two surveys and flow conditions had to be used.

TABLE 2

FLOW INPUTS TO MODEL

(All results in cfs unless noted)

	STATION	8/31/88	9/20/88
Quinebaug River above Sturbridge WWTP	QR01	22.4	21.1
Sturbridge WWTP	WWTP	0.529	0.529
Hamant Brook	QRT02	0.52	0.83
Hobbs Brook	QRT03	0.96	0.58
Unnamed tributary	QRT04	0.32	0.25
Breakneck Brook	QRT05	1.3	0.86
Hatchet Brook	QRT06	0.15	0.08
Incremental inflow (cfs/mile)	-	0.0	0.73
Westville Dam*	QR01E	26	28

* Not input

Incremental inflow was determined by subtracting all known inputs and the headwater flow from the Westville Dam flow and dividing by 4.6 miles, the distance from the headwaters of the model to the Westville Dam. This provided a constant inflow throughout the model in cfs/mile.

Flows were input and the model was run. The accuracy of the model was determined by comparing the predicted flow at the Westville Dam with the actual flow measurement recorded at the Westville Dam.

WIDTHS & DEPTHS

The average widths and depths for each reach were input into the model using the discharge coefficients a, b, and B in the equations $V = aQ^b$ and $D = \alpha Q^B$.

The discharge coefficients are determined using the velocity discharge plot (velocity vs. flow) and the stage discharge plot (depth vs. flow). Both graphs involve plotting (for each station) on log-log paper. Two or more depth or two or more velocity measurements are plotted on the y axis versus their corresponding flow measurement on the x axis. The average line for the measurements is drawn through the points. Where this line intercepts the one cfs point gives the value for the velocity "a" coefficient or the depth "α" coefficient. The slope of the average line is the "b" and/or "B" coefficients.

Width and depth coefficients were measured during the August and September 1988 surveys. These are listed in Tables 13 and 14 of the 1988 Data Report. The flows, depths and velocities were so similar and so close that an average line could not be drawn between them. Thus the a, b, α and B coefficients were estimated using the known flows, depths and velocities and an educated guess at the depths and velocities for a 7Q10 flow. There were no measurements taken during the 1988 survey in reaches 2, 3, and 5. Reach 2 was too deep to take a measurement (deeper than chest waiters), there were no stations or access to Reach 3, and Reach 5 was the Westville Impoundment where no measurements were taken either. The estimated discharge coefficients were input into the model and run at different flows. The calculated width and depths were checked to determine if the predictions were reasonable. Adjustments were made until the river profile for each reach at every flow was adequate.

The width and depth discharge coefficients used in the model are listed in Table 3.

TABLE 3

QUINEBAUG RIVER DISCHARGE COEFFICIENTS

REACH	a	b	α	B
1	0.129	0.2975	1.05	0.020
2	0.129	0.2975	2.50	0.004
3	0.240	0.5975	1.90	0.020
4	0.350	0.596	0.95	0.168
5	0.020	0.006	5.05	0.010
6	0.480	0.178	0.125	0.557

Coefficients are used in the equations $V = aQ^b$ and $D = \alpha QB$.

REACTION COEFFICIENTS Kd, Kn, Kp

The BOD decay coefficient (Kd), Nitrogenous decay coefficient (Kn) and phosphorus decay coefficient (Kp) are all estimated using the same equation and instream field data. The coefficients are estimated using the equation:

$$K_x = \frac{\ln (Xa/Xb)\ 86400\ V}{L}$$

Where:

Kx = reaction coefficient for parameter x (1/day)
Xa = BOD, NH_3 or TP concentration at upstream station
Xb = corresponding BOD, NH_3 or TP concentration at downstream station
V = velocity in feet/sec
L = distance between stations in feet

The number 86400 converts seconds to days.

Rate coefficients and reaction rates vary with temperature. The Qual2E model requires coefficient values be input at 20°C (68°F) and the model adjusts the K_2 value according to the river's temperature by the equation:

$$(K_2)_T = (K_2)_{20}\ \ O^{T-20}$$

Where:

$(K_2)_T$ = reaeration rate at T°C (1/day)
$(K_2)_{20}$ = reaeration rate at 20°C (1/day)
O = 1.024
T = temperature (°C)

The August 1988 survey Biochemical oxygen demand (BOD), ammonia-nitrogen (NH_3) and phosphorus (P) data was used to determine the decay coefficients. These data can be found in the 1988 Quinebaug report. An example of a BOD decay coefficient calculation is included below:

Stations QR01B QR01C - Reach 3

$$kd = \frac{\ln (1.2\ mg/l/0.9\ mg/l)\ 86400s/day \times 0.17fts}{1.5\ mg/l\ (5280\ ft/mile)} = 0.534\ day^{-1}$$

The temperature was approximately 20°C so no temperature adjustments were made.

In Reach 5, the impoundment, BOD, NH_3 and P all increased in concentration and thus a decay value could not be determined. The increase is either a result of an unknown discharge contributing elevated concentrations or from algal interference in the BOD and ammonia calculations.

Once the reaction rates are estimated using the first equation, they are adjusted to 20°C and input into the computer model. The model was run, and the model's BOD, ammonia and phosphorus estimates were compared to actual instream data. If results did not have an accurate fit, the reaction rates were adjusted until the fit was good.

The final reaction rate coefficents input to the model are presented in Table 4. The decay coefficients calculated from the water quality data are listed in parenthesis before the actual adjusted value used in the model.

TABLE 4

REACTION RATE COEFFICENTS (1/day)

REACH	Kd		Kn		Kp	
1	(2.88)	3.080	(1.406)	1.41	(3.15)	3.55
2	(2.26)	3.080	(4.114)	2.49	(0.88)	1.50
3	(0.534)	0.534	(0.83)	0.83	(0.26)	0.17
4	(2.212)	2.210	(0.575)	0.58	(0.58)	0.29
5	(0.0)	0.000	(0.0)	0.00	(0.0)	0.00
6	(1.75)	1.700	(0.0)	0.00	(9.74)	1.90

REAERATION (K_2) AND SEDIMENT OXYGEN DEMAND

BOD and ammonia oxidation remove oxygen from the stream. The natural flow of the river and oxygen transfer from the atmosphere replace some of the consumed oxygen back into the stream. Several different scientists have studied the reaeration phenomenon and developed equations to estimate the reaeration based on the physical characteristics (depth and velocity) of the river. In 1976 a scientist named Covar made a plot based on stream depth and river velocity containing reaeration equations by three scientists: O'Connor, Owens and Churchill (Tetra Tech 1978). The plot recommends that for stream depths less than two feet and any velocity, the Owens reaeration equation should be used, and for stream depths greater than two feet and velocities less than one foot/ second, the O'Connor reaeration equation should be used. The Churchill equation should be used for velocities greater than one foot per second and depths greater than two feet. This plot is only to aid in choosing a reaeration equation. The Qual2E model, however, also calculates reaeration using equations created by Thackston and Krenkel, Lanbien and Durum, Tsivoglou-Wallace and an enter your own version.

The EPA Tetra Tech Rate Manual lists the conditions each scientist used to develop their reaeration equation. The conditions (flow velocities and depth) varied from scientist to scientist. The reaeration equation, therefore, should be chosen based on similarity between the stream conditions and experimental conditions. The majority of the Quinebaug River had depths less than one foot/sec. From Covars plot, the Owens equation had the most similar instream conditions. The Tsivoglou equation was derived from rivers with flows between 25 and 3000 cfs, the Langbien & Durum equation was based on data from O'Connor and Dobbins, Churchill, Krenkel and Orlob, and Streeter (all obtained under deeper river water conditions) and no stream conditions were provided for the Thackston and Krenkel equation. Thus, the Owens reaeration equation was chosen for input to the model since it was the most similar.

After BOD decay, and nitrogenous decay coefficients were adjusted and determined to be accurate, the choice of reaeration equation was input into the model. The model then calculates the reaeration coefficient based on the streams depth, velocity and water temperature. Since all the decay and reaeration coefficients are assumed to be correct, the sediment oxygen demand (SOD) value is adjusted until the predicted instream DO calculations match with the actual observed dissolved oxygen.

Different reaeration equations should be tried and their corresponding SOD's compared. Sediment oxygen demands in a river range from 0-10 $gm/m^2/day$, with SOD concentrations of 1-3 $gm/m^2/day$ being the most common. SOD is a measure of the oxygen utilized by benthic deposits of organic material and attached bacterial growth. The different SOD's determined from the different reaeration equations can be evaluated to determine which set of SOD's are the most realistic for the particular stream. The SOD rates and K_2's used in the Quinebaug River model are listed in Table 5.

Reaches 2 and 3 have higher SOD values than Reach 1 located directly below the WWTP's discharge. Reach 2 has a deeper section with slower flows than the reach directly below the plant. More solids settle in slower more sluggish sections.

One would assume the Westville Impoundment would have higher SOD's than the river since settling of solids would occur there. The values, however, were very low. This could be from solids settling upstream and the lack of a nearby discharge.

TABLE 5

SOD AND LOW FLOW REAERATION VALUES

Reach	SOD gm/ft^2/day	SOD gm/m^2/day	REAERATION (Owens eq.) K_2 1/DAY
1	0.150	1.63	7.20
2	0.250	2.72	1.63
3	0.205	2.23	1.21
4	0.050	0.55	3.11
5	0.035	0.38	0.16
6	0.650	7.08	1.79

CALIBRATION AND VERIFICATION

Once all the width and depth, flow, reaction and reaeration coefficients were determined they were input to the computer with the August 31, 1988 water quality data. The Qual2E model was run and the predicted data were compared to the actual instream data. Coefficients were adjusted until the model predicted the actual data fairly accurately. Once the model was calibrated the second set of instream data (September 20, 1988) was input. All width and depth, reaction coefficients and reaeration coefficients were kept constant while adjustments were made to the tributary flows, incremental inflows and water quality of the tributaries, wastewater effluent and Quinebaug River headwater data.

The predicted river water quality data from the September 20 inputs were compared to the actual observed survey data. If the predicted data were very different from the actual data, then the corresponding coefficients were adjusted until the actual data were predicted fairly accurately. The new coefficients would then be run through the original August 31, 1988 data model to make sure the original model still predicted the original data reasonably.

Eventually, a final model was verified that predicted both sets of survey data fairly accurately. Model predictions vs actual data for BOD, NH_3, TP and D.O. are plotted in Figures 3 - 10. Dissolved oxygen concentrations were modeled to simulate the average observed dissolved oxygen. Photosynthetic activity in the stream caused variations in dissolved oxygen concentrations, so the model was calibrated to predict a dissolved oxygen value between the minimum and maximum observed values.

BOD and ammonia-nitrogen concentrations both increased as the river flowed through the Westville Impoundment. The BOD increase could be from the increased algae in the impoundment that caused the photosynthetic activity and the NH3 increase could be from the conversion of TKN data to ammonia. This increase from algae could not be predicted in the Qual2E model and thus the BOD and NH_3 predictions in the last 2 reaches are not predicted accurately. (The Kd coefficient was set equal to zero in reach 5 and the kn coefficient in reaches 5 and 6 since there was no decrease.)

FIGURE 3

STURBRIDGE WLA

MODEL PREDICTION VS ACTUAL BOD 8/31/88

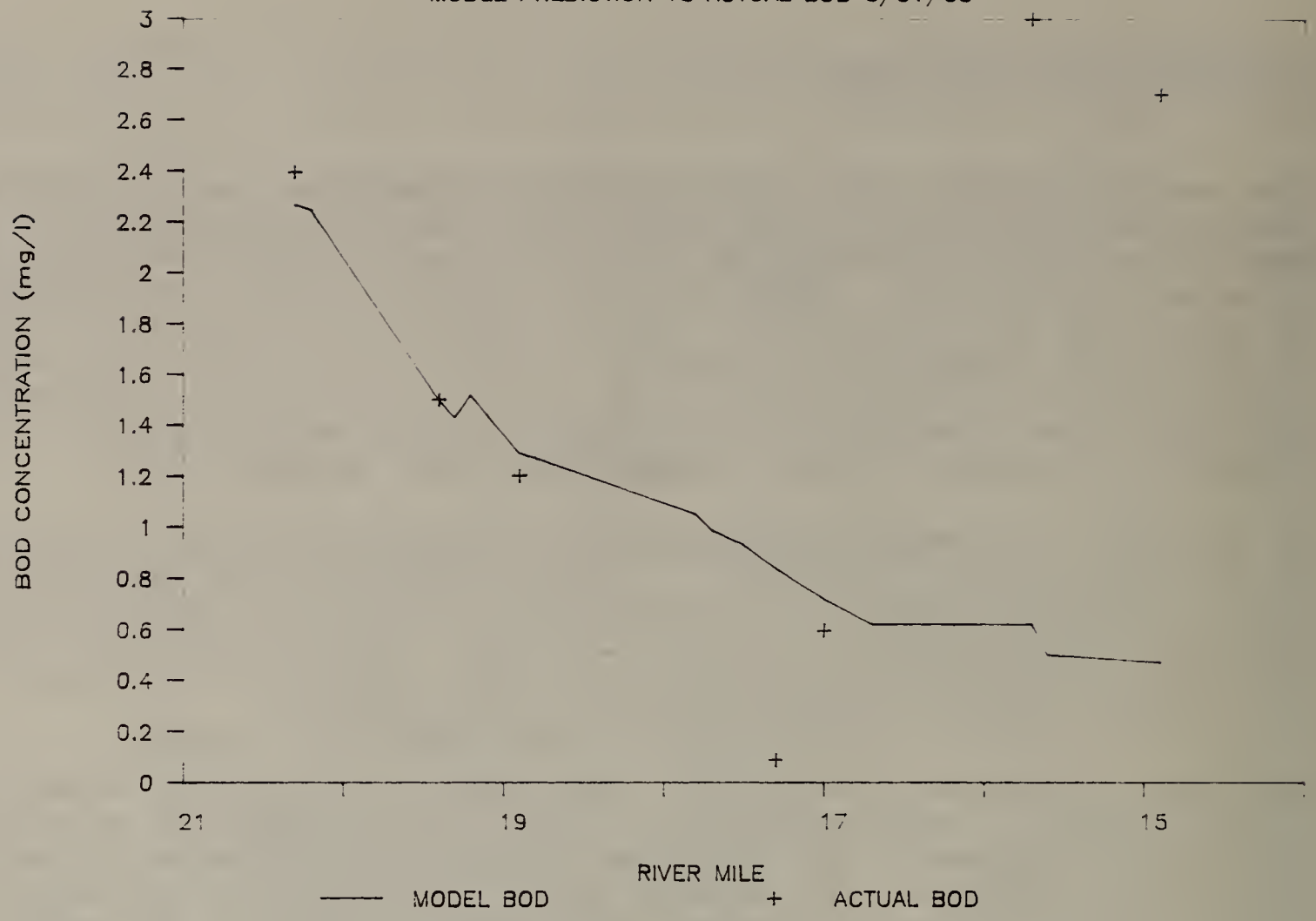


FIGURE 4

STURBRIDGE WLA

MODEL PREDICTION VS ACTUAL NH3 8/31/88

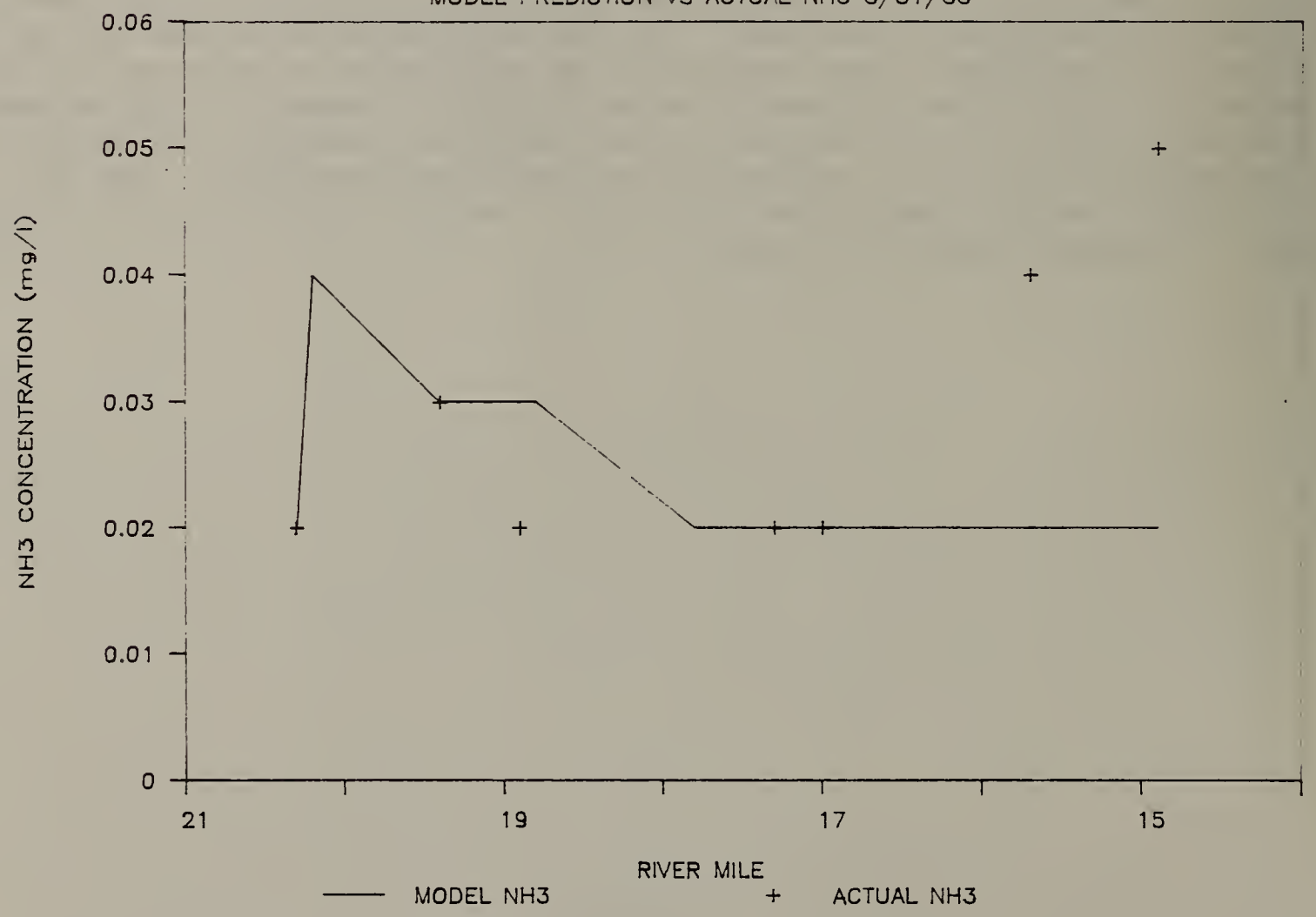
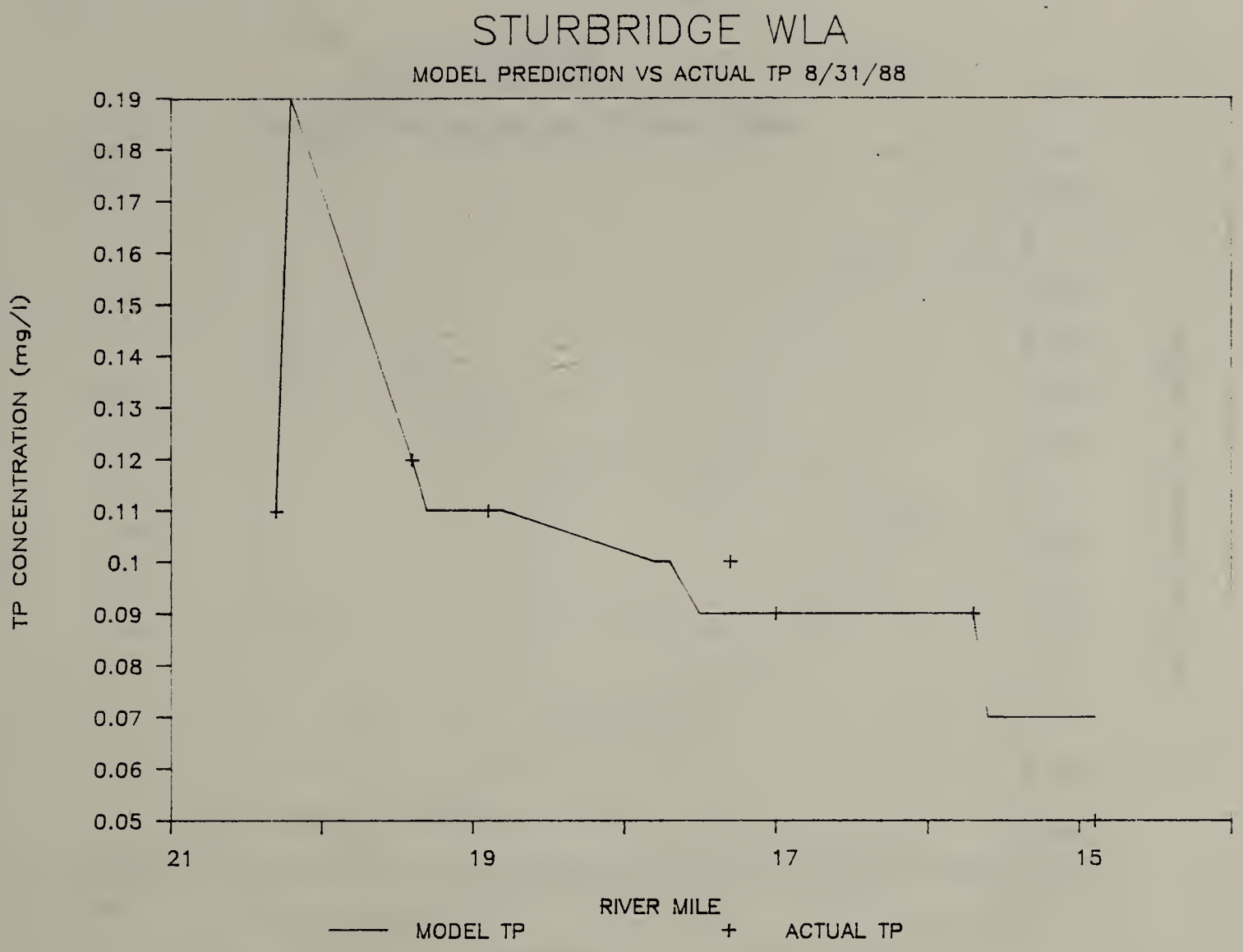


FIGURE 5



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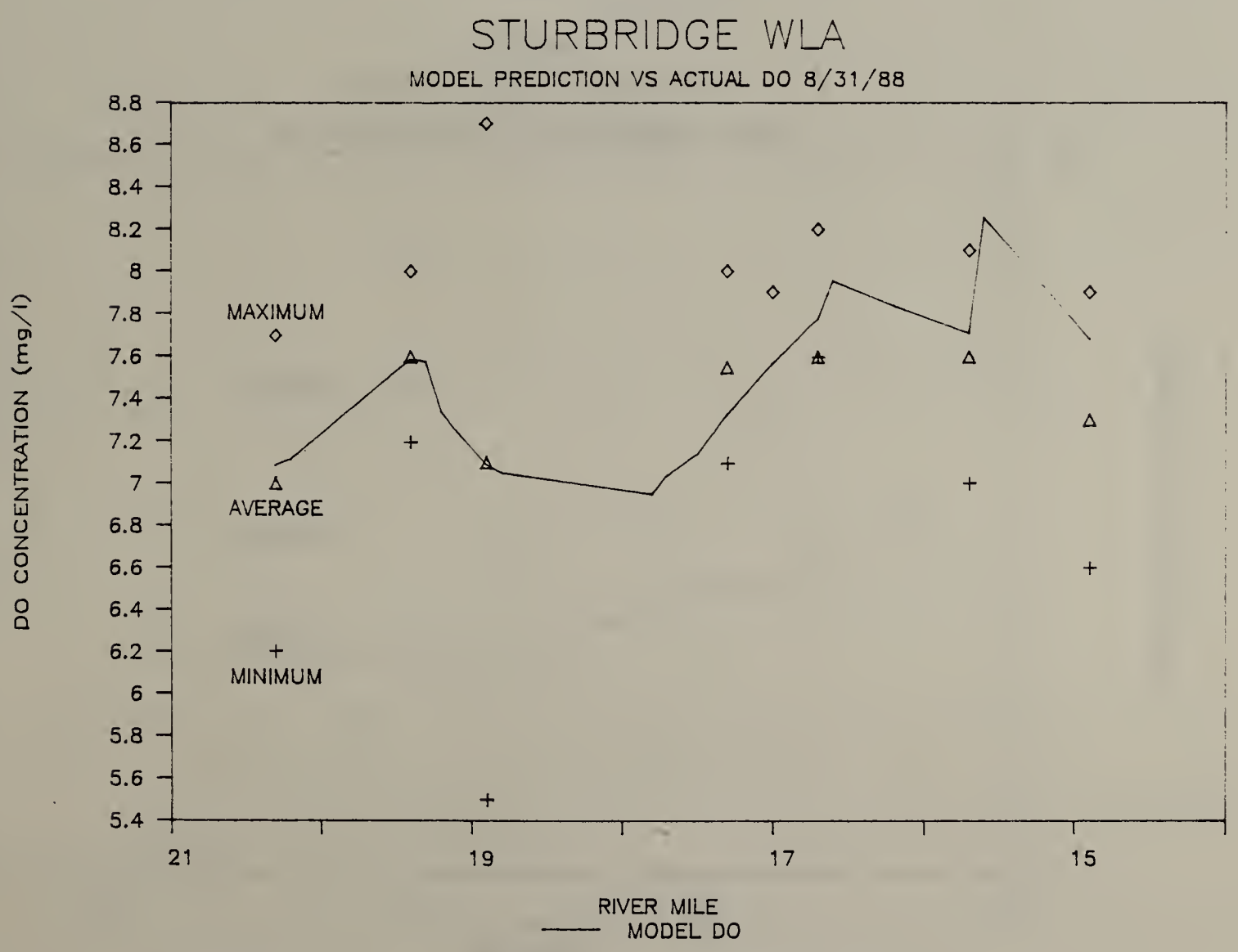

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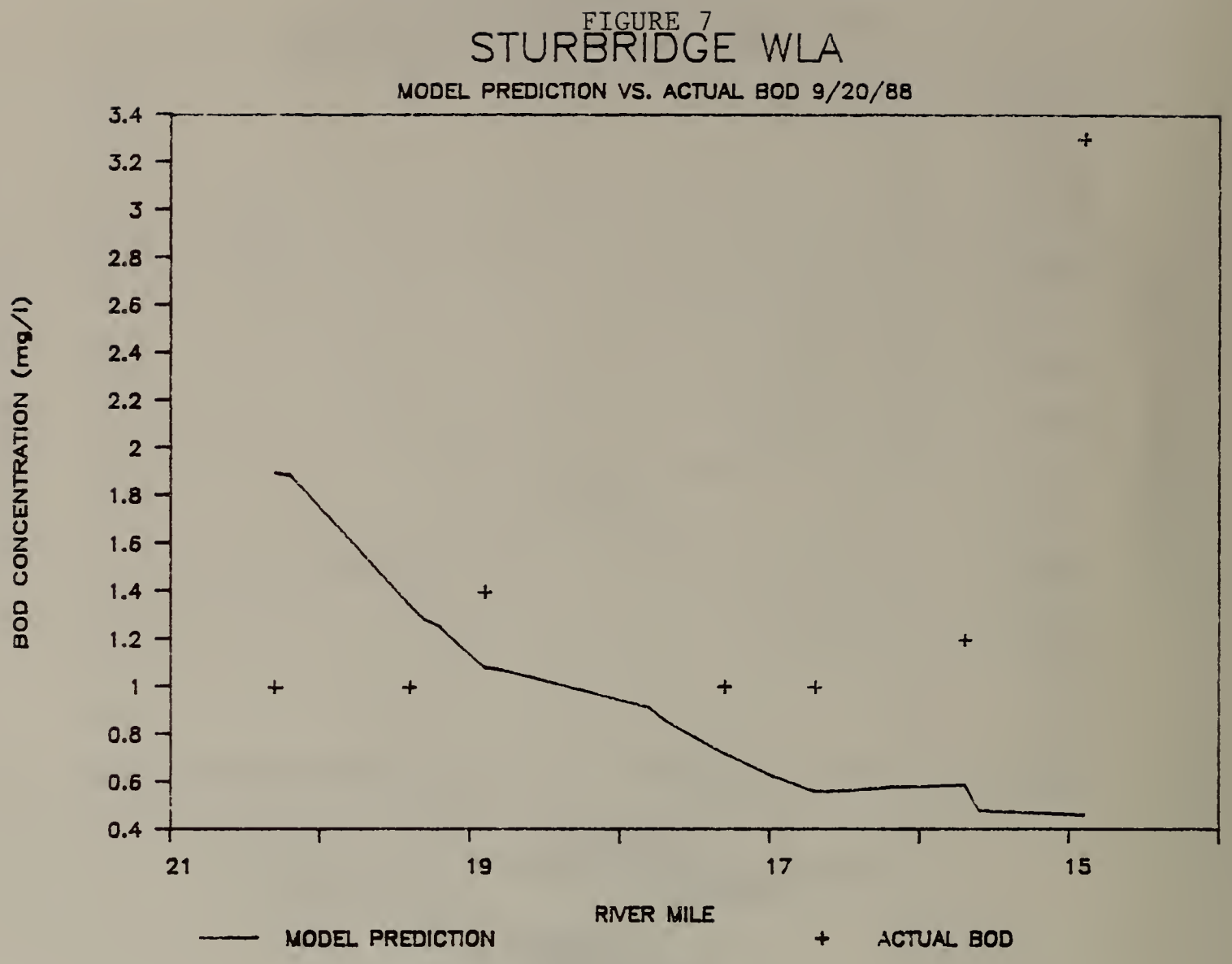


FIGURE 8

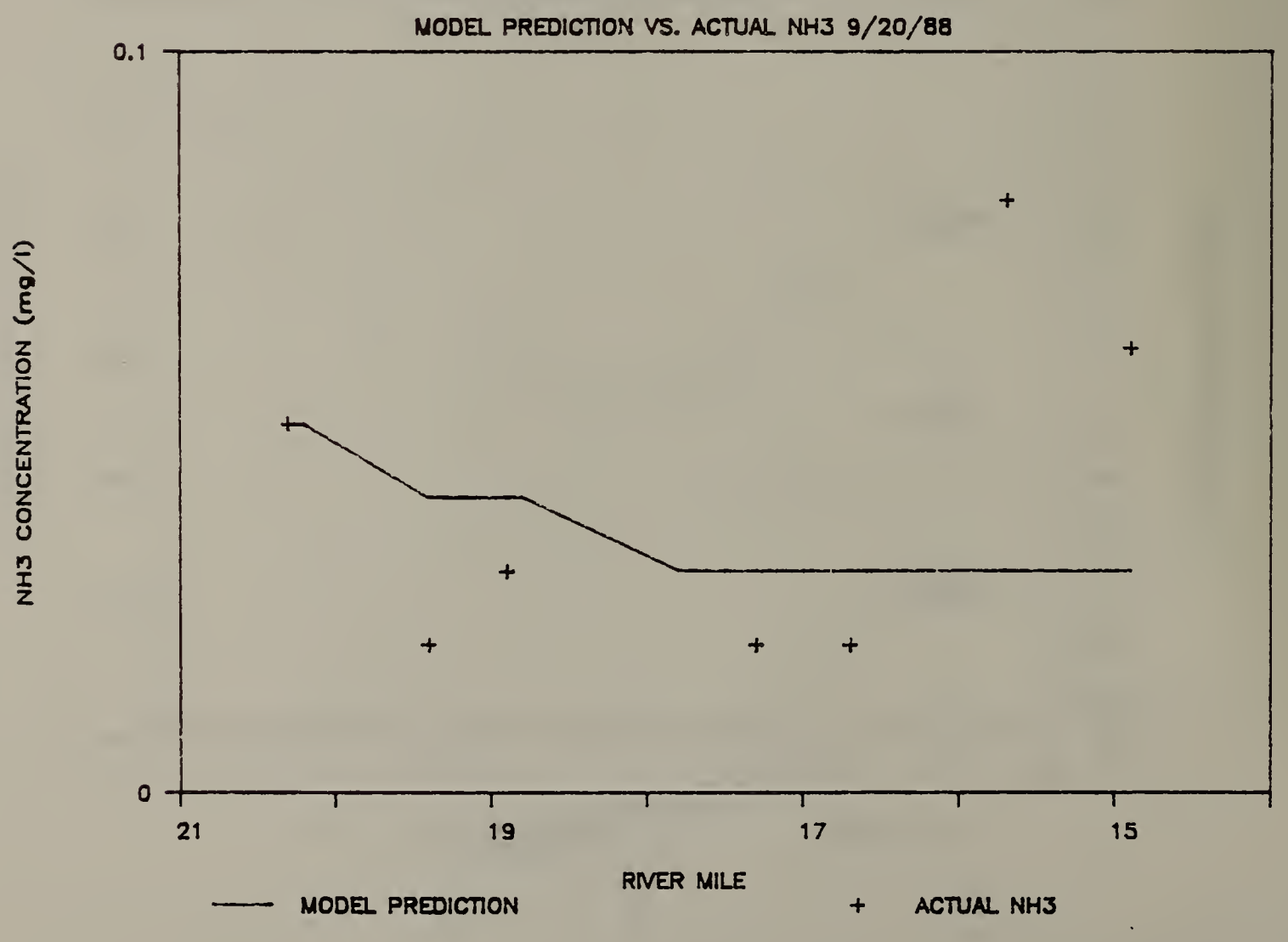


FIGURE 9

STURBRIDGE WLA

MODEL PREDICTION VS. ACTUAL TP 9/20/88

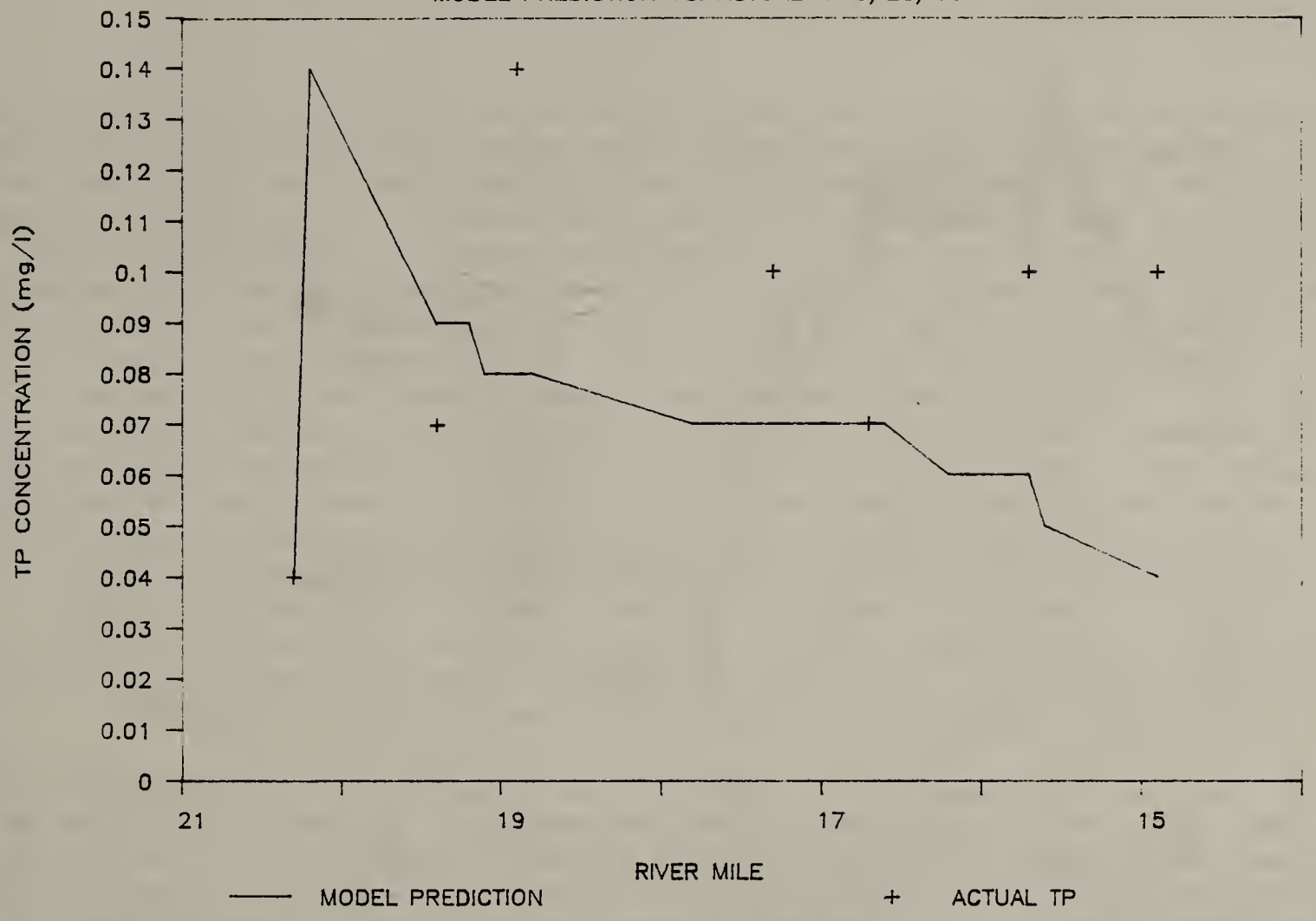
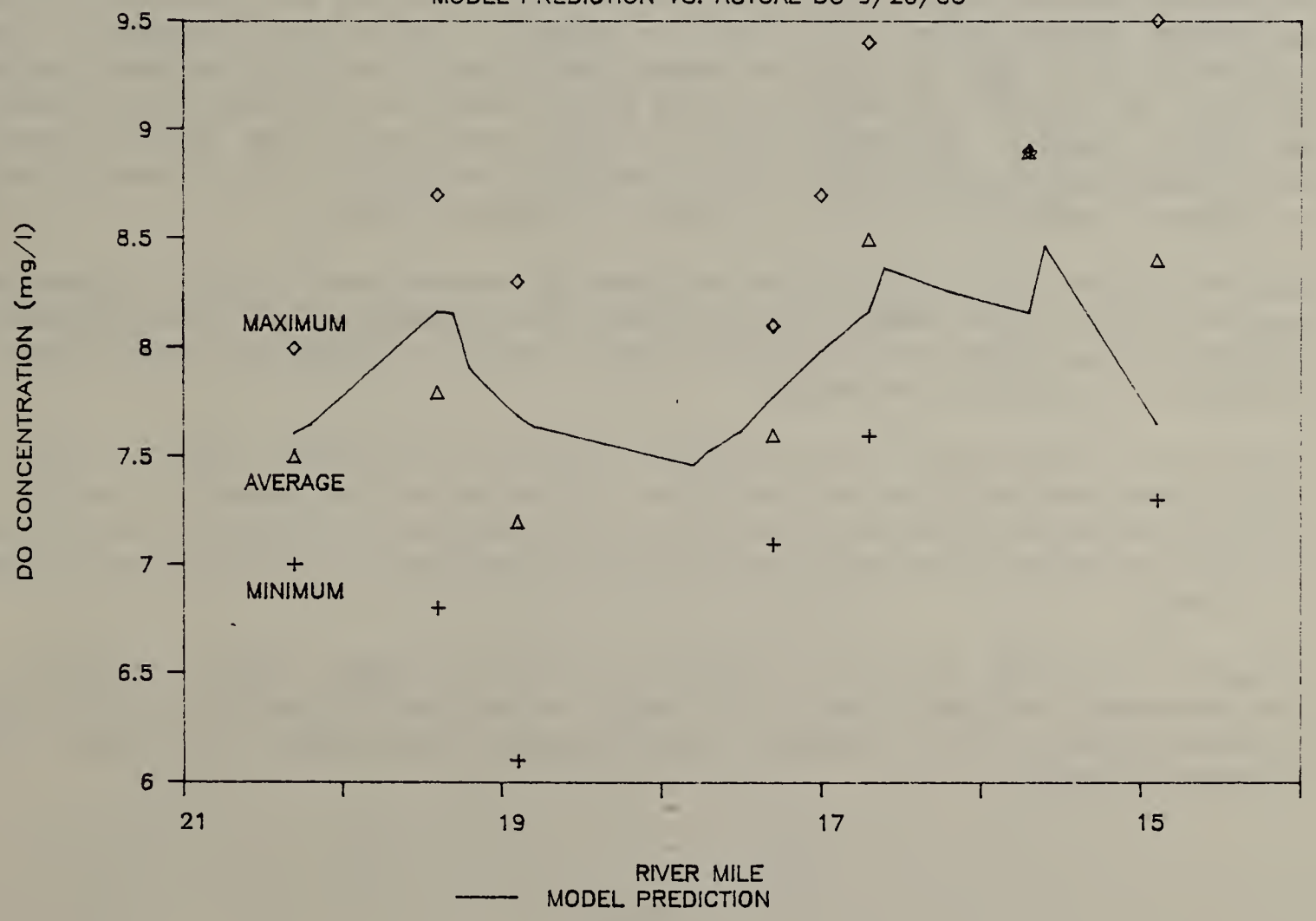


FIGURE 10

STURBRIDGE WLA

MODEL PREDICTION VS. ACTUAL DO 9/20/88



LOW FLOW ANALYSIS

Once the model was calibrated and verified to two different sets of data, the model was changed to low flow conditions. Water temperatures were set at 70°F and a 7Q10 flow of 6.82 cfs was input for the Quinebaug River flow just above the Sturbridge WWTP. Tributary 7Q10 flows were assumed to be 0.1 cfs. An assumed flow of 0.1 cfs is an estimate. Streams with similar characteristics and drainage areas were listed in the U.S. Geological Surveys Gazetteer as having estimated 7Q10 low flows of 0 to 0.3 cfs. The majority of the tributaries had 7Q10 flows equal to or less than 0.1 cfs. The WWTP NPDES permit limits of 0.5 MGD (0.77 cfs) and 30 mg/l for BOD were input. Ammonia-nitrogen in the effluent was assumed to be 4.1 mg/l. Tributary concentrations were held equal to the August data. These limits were much higher than was observed during any of the previous wastewater effluent surveys. With low flow conditions and the secondary effluent permit limits the model predicted the instream dissolved oxygen would decrease to 3.9 mg/l 2.4 miles downstream of the Sturbridge WWTP discharge. Limits of 20 mg/l for BOD and 3 mg/l for NH_3 are needed for the 0.5 MGD plant to allow water quality standards to be met instream below the discharge (Figure 11). The plant is currently operating well enough to meet these stricter limits and hence not violate water quality standards.

The low flow model predicted a large dissolved oxygen sag in reach 3, one and a half miles below the Sturbridge WWTP discharge. This section of river is wider, with larger flood plains and no elevation drop. A review of all the dissolved oxygen data from past surveys does not show this D.O. sag (see Table 6 for comparisons). There are many reasons for this lack of proof. First of all, no water quality samples have ever been taken from this stretch of river. Second, the past surveys have all been conducted when flows in the Quinebaug River have been three or more times as great as the 7Q10 low flow. Finally, the Sturbridge Wastewater Treatment plant has been operating well below its secondary NPDES permit limits. In 1988, for example, the effluent BOD was at concentrations of 7 mg/l or less and ammonia-nitrogen concentrations were below 1 mg/l; 25% of the BOD and NH_3-N loading which were input for low flow conditions and permit limits. The 1988 survey was also conducted during the fall when water temperatures were cooler (than low flow conditions) and DO concentrations are higher (saturation level is inverse to temperatures).

Thus, while the 1988 surveys (and all others) showed water quality standards were met easily, with warmer temperatures, four times the effluent pollutant loading and one third the river flow, water quality violations would occur as predicted by the low flow model.

Since the low flow model was reasonable, different effluent flows and concentrations were input to determine the limits which are needed for water quality standards to be met. The Town of Sturbridge is requesting an increase in their wastewater flow from their current capacity of 0.5 MGD up to a capacity of 1.0 MGD.

Initially, effluent flows of 1.0 MGD (1.55 cfs) was input into the low flow model with secondary effluent limits of 30 mg/l for BOD and SS and 10 mg/l for NH_3. The model predicted the dissolved oxygen concentration would drop below 3 mg/l with these limits.

FIGURE 11

0.5 MGD

EFFLUENT VARIATIONS OF BOD AND NH3

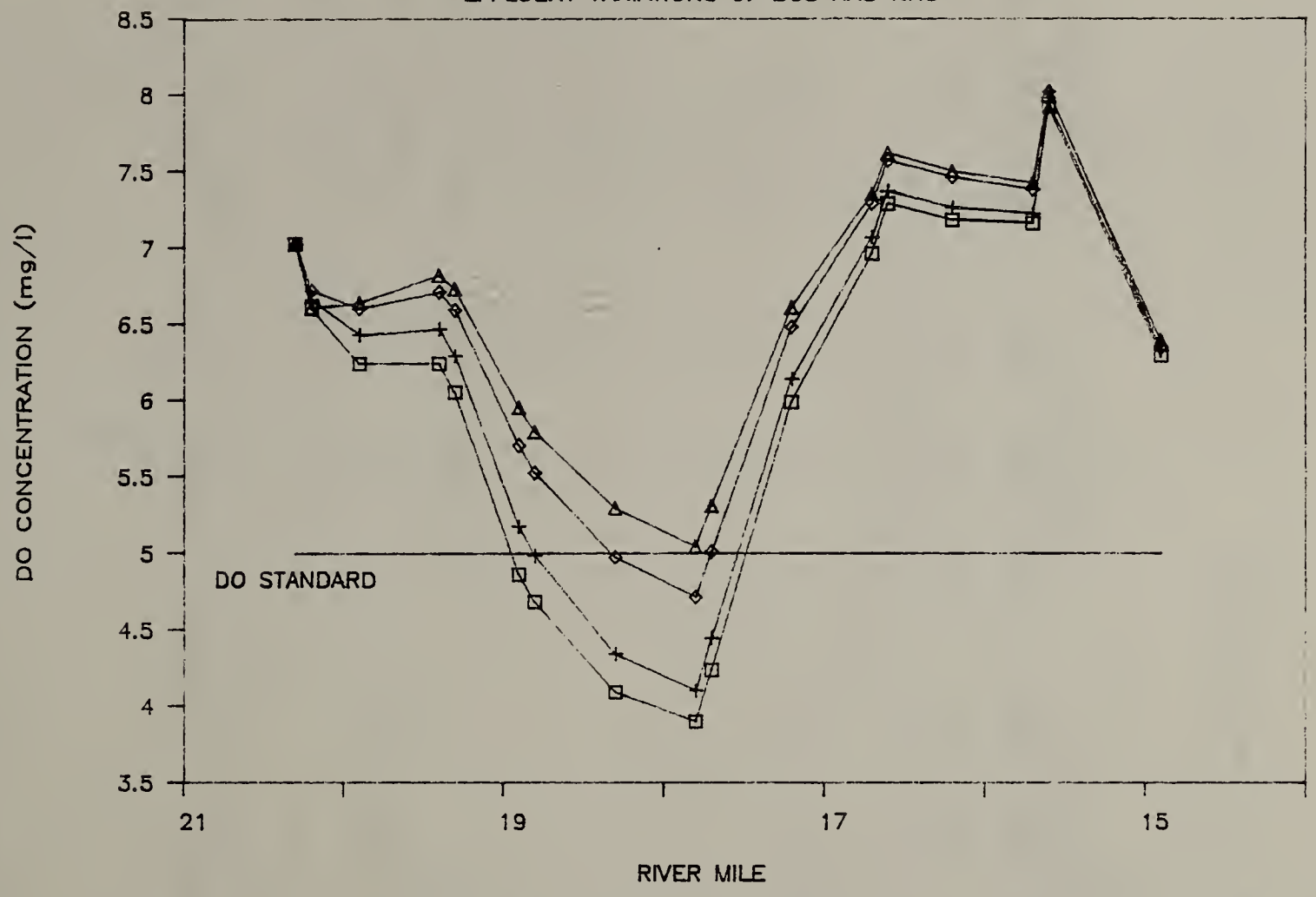


TABLE 6

COMPARISONS OF PAST SURVEYS D.O. AND FLOW DATA

PREVIOUS STATION NUMBER	1988 STATION NUMBER	RIVER MILE	8/22-25/72			7/30-8/2/74			6/29-7/1/76			9/14-16/76		
			MIN.	MAX.	AVG.	MIN.	MAX.	AVG.	MIN.	MAX.	AVG.	MIN.	MAX.	AVG.
QR01	-	21.3	5.9	9.1	7.7	6.5	8.6	7.5	6.6	9.3	7.6	7.1	9.0	8.3
-	QR01	20.5	-	-	-	-	-	-	-	-	-	-	-	-
QR01A	QR01A	19.4	-	-	-	6.1	10.5	7.6	5.6	9.0	7.1	7.5	9.8	8.2
QR01B	QR01B	18.9	-	-	-	-	-	-	6.6	8.3	7.2	7.8	9.3	8.4
-	QR01C	17.4	-	-	-	-	-	-	-	-	-	-	-	-
-	QR01D	16.7	-	-	-	-	-	-	-	-	-	-	-	-
-	QR01E	15.7	-	-	-	-	-	-	-	-	-	-	-	-
QR01C	QR01F	14.9	-	-	-	-	-	-	7.0	7.6	7.4	6.6	10.3	8.8
QR02	-	13.6	-	-	-	6.6	8.9	8.1	6.8	9.5	7.7	8.6	10.0	9.0
FLOW	QR01	21.3 or 20.5	-	-	-	-	12.6	-	-	22.6	-	38.7	43	40
	Westville Dam	15.7	-	30	-	-	22	-	-	30	-	-	30	-

TABLE 6 (CONTINUED)

PREVIOUS STATION NUMBER	1988 STATION NUMBER	RIVER MILE	4/17/85	6/24/85	6/25/85	8/31/88 MIN.	8/31/88 MAX.	9/20/88 MIN.	9/20/88 MAX.
QR01	-	21.3	10.7	7.7	7.9	-	-	-	-
-	QR01	20.5	-	-	-	6.2	7.7	7.0	8.0
QR01A	QR01A	19.4	10.4	7.4	8.2	7.2	8.0	6.8	8.7
QR01B	QR01B	18.9	-	-	-	5.5	8.7	6.1	8.3
-	QR01C	17.4	-	-	-	7.1	8.0	7.1	8.1
-	QR01D	16.7	-	-	-	7.6	8.2	7.6	9.4
-	QR01E	15.7	-	-	-	7.0	8.1	8.9	8.9
QR01C	QR01F	14.9	-	-	-	6.6	7.9	7.3	9.5
QR02	-	13.6	10.0	8.2	8.6	-	-	-	-
FLOW	QR01	21.3 or 20.5	-	-	-	22.4	-	21.1	-
	Westville Dam	15.7	85	40	69	26	-	28	-

Different BOD and NH_3 concentrations were entered until the effluent caused instream water quality standards to be met. Effluent BOD concentrations of 30, 20 and 10 mg/l were input. Ammonia-nitrogen concentrations were initially set at 10 mg/l and then lowered to 5 and 2 mg/l. Effluent dissolved oxygen (D.O.) concentrations were varied between 5 and 6 mg/l.

From the model's predictions it appears a small ammonia-nitrogen removal is more beneficial in D.O. improvement than a small BOD removal. A 2 mg/l decrease in ammonia improves the water quality as much as a 10 mg/l decrease in BOD. This is consistent with the chemical equilibrium equation where 4.57 mg/l of oxygen are required to breakdown one mg/l of ammonia-nitrogen. Likewise, since BOD is a measure of oxygen required for biological breakdown, one mg/l of oxygen is consumed by one mg/l of BOD. These two equations can be combined so that 2 mg/l of ammonia are approximately equal to 10 mg/l of total oxygen demand (or BOD). This is similar to the model's determination. Also, an effluent D.O. of 6 mg/l (instead of 5 mg/l) will improve the D.O. immediately below the WWTP by only a small amount (0.1 mg/l) which later eventually equalizes with the 5 mg/l D.O. effluent predictions. Thus effluent D.O. concentrations were held at 5 mg/l for all modeling runs.

The final recommended limits for the Sturbridge WWTP 1.0 MGD discharge are 10 mg/l for BOD and SS and 2 mg/l for NH_3-N.

An effluent flow of 0.75 MGD (1.16 cfs) and its required limits were also evaluated. Different BOD and NH_3 concentrations were input until the model predicted that designated limits would cause class B water quality standards would be met. Recommended limits for a 0.75 MGD effluent are 10 mg/l for BOD and SS and 3 mg/l for NH_3. Results are plotted in Figures 12 and 13.

Effluent phosphorus concentrations were also evaluated. The phosphorus decay coefficients were determined in the models creation and verification runs. During low flow conditions three different phosphorus concentrations were tried in the effluent. Concentrations of 4.2 and 1 mg/l for 0.75 MGD and 1.0 MGD are plotted in Figures 14 and 15. The EPA recommended concentrations for phosphorus in flowing streams to prevent eutrophication is 0.1 mg/l. For both a 0.75 MGD and 1.0 MGD wastewater flow, a phosphorus effluent concentration of 1.0 mg/l is recommended.

FIGURE 12

0.75 MGD

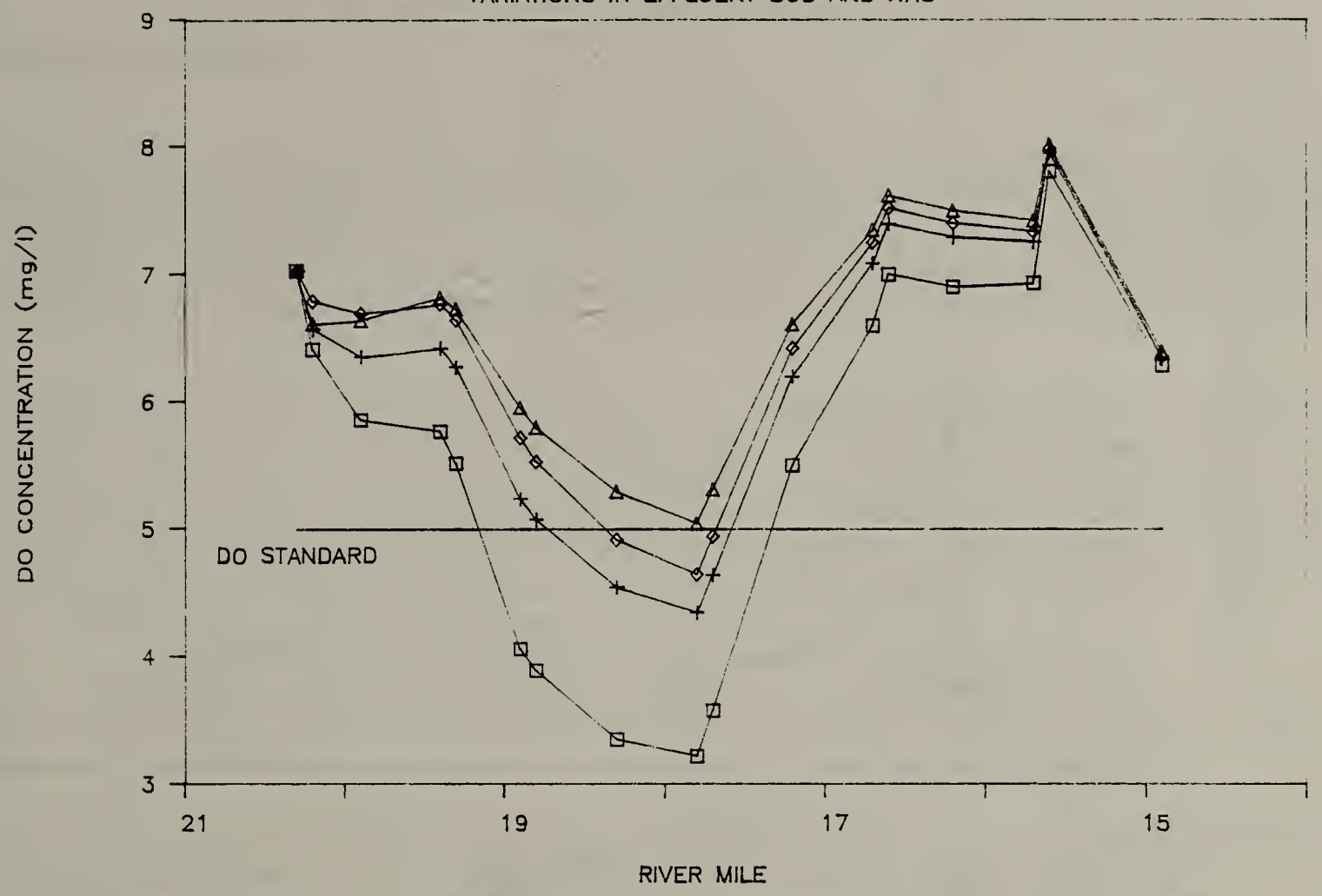


FIGURE 13

1.0 MGD

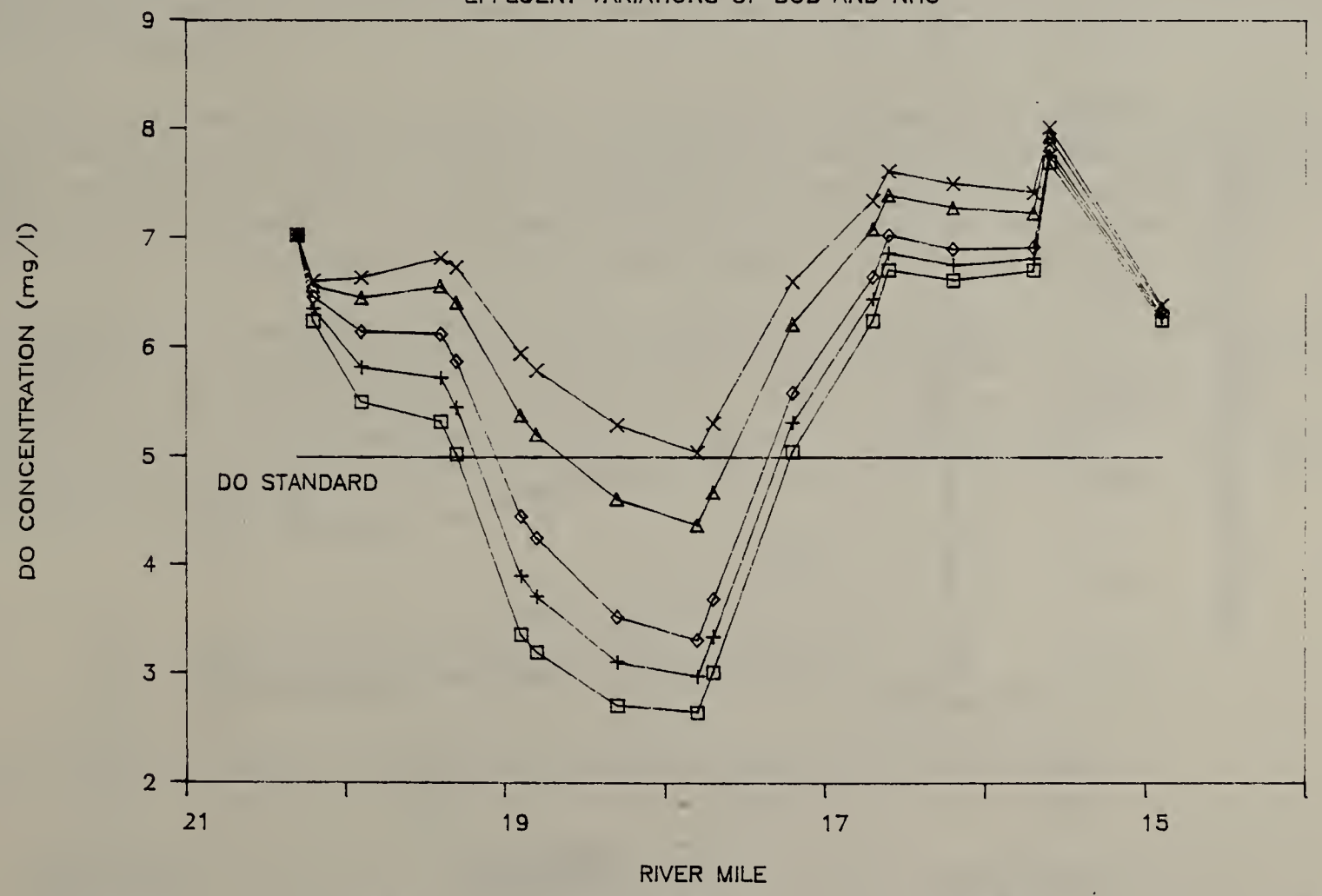


FIGURE 14

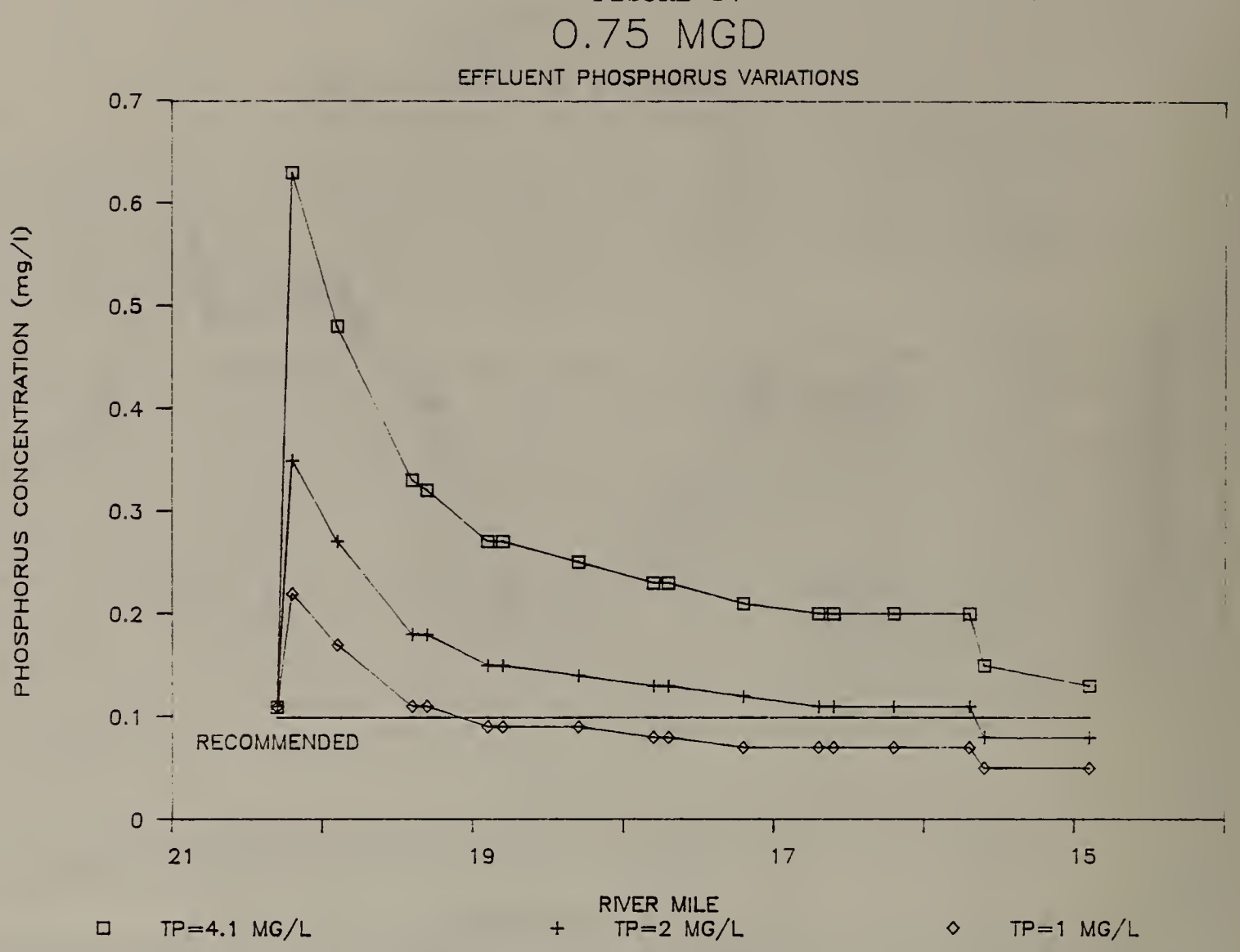
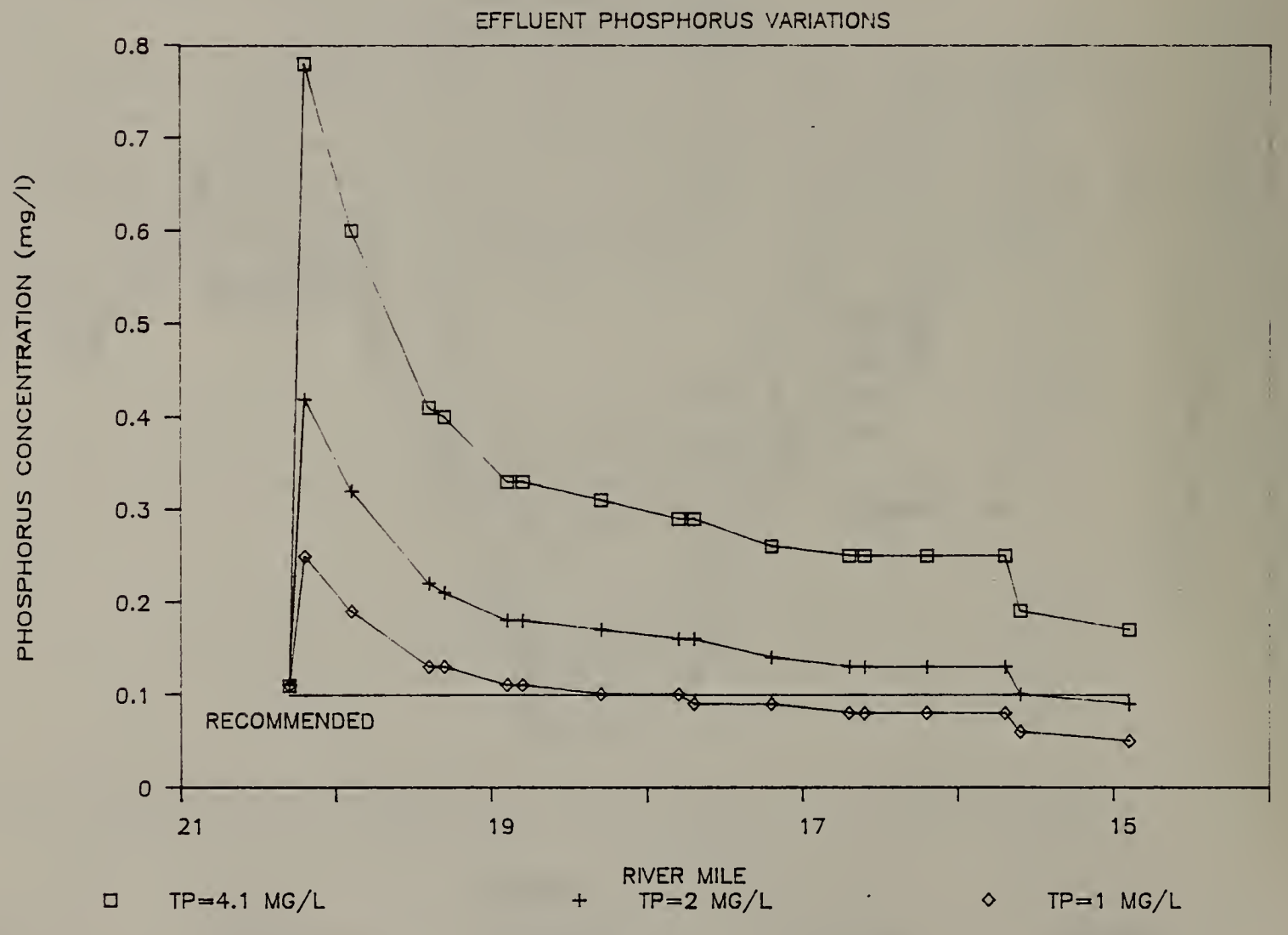


FIGURE 15



SENSITIVITY ANALYSIS

When varying the BOD and ammonia-nitrogen concentrations in the effluent, it was observed that a 20 mg/l decrease in BOD increased the instream dissolved oxygen approximately the same amount as a three (3) mg/l decrease in ammonia-nitrogen. The ammonia seemed to have more of an effect on D.O. concentration than BOD. Both BOD decay and nitrogenous decay coefficients (Kd and Kn) thus were varied and run through the model for sensitivity analysis.

A sensitivity analysis was run using the low flow model by varying two different reaction coefficients (Kd and Kn) and noting the change in predicted instream dissolved oxygen. The BOD decay coefficient (Kd) and the nitrogenous decay coefficient (Kn) were varied to three different increments and input into the Qual2E model. The three values included the original low flow value, half the original value (0.5 Kd, 0.5 Kn) and one and a half times the original value (1.5 Kd, 1.5 Kn). The different inputs are listed in Table 7.

Each of these values was input to the low flow model (1.0 MGD, BOD = 10 mg/l, NH_3 = 2 mg/l) to predict the instream dissolved oxygen concentrations. (See Figures 16 and 17).

The sensitivity analysis predictions (Figures 16 and 17) for Kd and Kn did not show large instream D.O. concentration variations with 50% coefficient variations. The Kn coefficients in particular, caused very little D.O. variation. The Kn coefficient caused the greatest variation (0.3 mg/l) in the first mile and a half below the WWTP discharge. This change is minimal. At the lowest point of the D.O. sag there was a maximum difference of 0.16 mg/l in the predicted dissolved oxygen. Thus, the BOD and nitrogen decay coefficients are fairly accurate and will not cause drastic differences in the predicted instream dissolved oxygen.

The other parameters of concern for the sensitivity analysis are reaeration and SOD. Reaeration, like Kd and Kn, is a reaction coefficient raised to a power. There are numerous equations for predicting reaeration. The Owens equation was used during the models creation and verification since it was created under river conditions and characteristics most similar to the Quinebaug River.

To evaluate the reaeration sensitivity, the Owens K_2 equation was tried at 50% and 150% of the original value. In addition, a third value of 2/H was input to the model. The 2/H reaeration coefficient is often used as a conservative reaeration (K_2) value used as a default. (H is the depth in feet). The reaeration values are listed in Table 8 and predicted instream dissolved oxygen is plotted in Figure 18.

SOD sensitivity analysis was also conducted. SOD can have a dramatic affect on the instream dissolved oxygen concentrations. In the basic Streeter Phelps equation the Kn, Kd and K_2 (reaeration) are all coefficients raised to a power to describe a rate of oxygen consumption. The SOD, however, is the actual measure of oxygen consumed by benthic deposits and attached bacterial growth. SOD concentrations are thus subtracted directly in the equation, and have a larger impact on the instream dissolved oxygen than a change in the decay coefficients. The sensitivity analysis for SOD is thus different than the sensitivity analysis for the reaction rates.

The SOD values were the last values adjusted in the model. They are based on the assumption that all the other coefficients are correct. There is no actual measured SOD values available for the Quinebaug River. If SOD concentrations appear high in some river reaches, it may be a result of the abundant plant and algae growth along the river bottom. The SOD is a measure of all the unknown different oxygen sources and sinks present in the system that have not been accounted for.

TABLE 7

SENSITIVITY ANALYSIS

VARIATIONS OF Kd AND Kn FOR INPUT TO QUAL2E

BOD Decay (Kd)

REACH	0.5Kd	Kd	1.5Kd
1	1.540	3.08	4.62
2	1.540	3.08	4.62
3	0.267	0.534	0.801
4	1.107	2.21	3.315
5	0.000	0.00	0.00
6	0.850	1.70	2.55

Nitrogenous Decay Kn

REACH	0.5Kn	Kn	1.5Kn
1	0.705	1.41	2.115
2	1.245	2.49	3.735
3	0.415	0.83	1.245
4	0.290	0.58	0.870
5	0.000	0.00	0.000
6	0.000	0.00	0.000

FIGURE 16

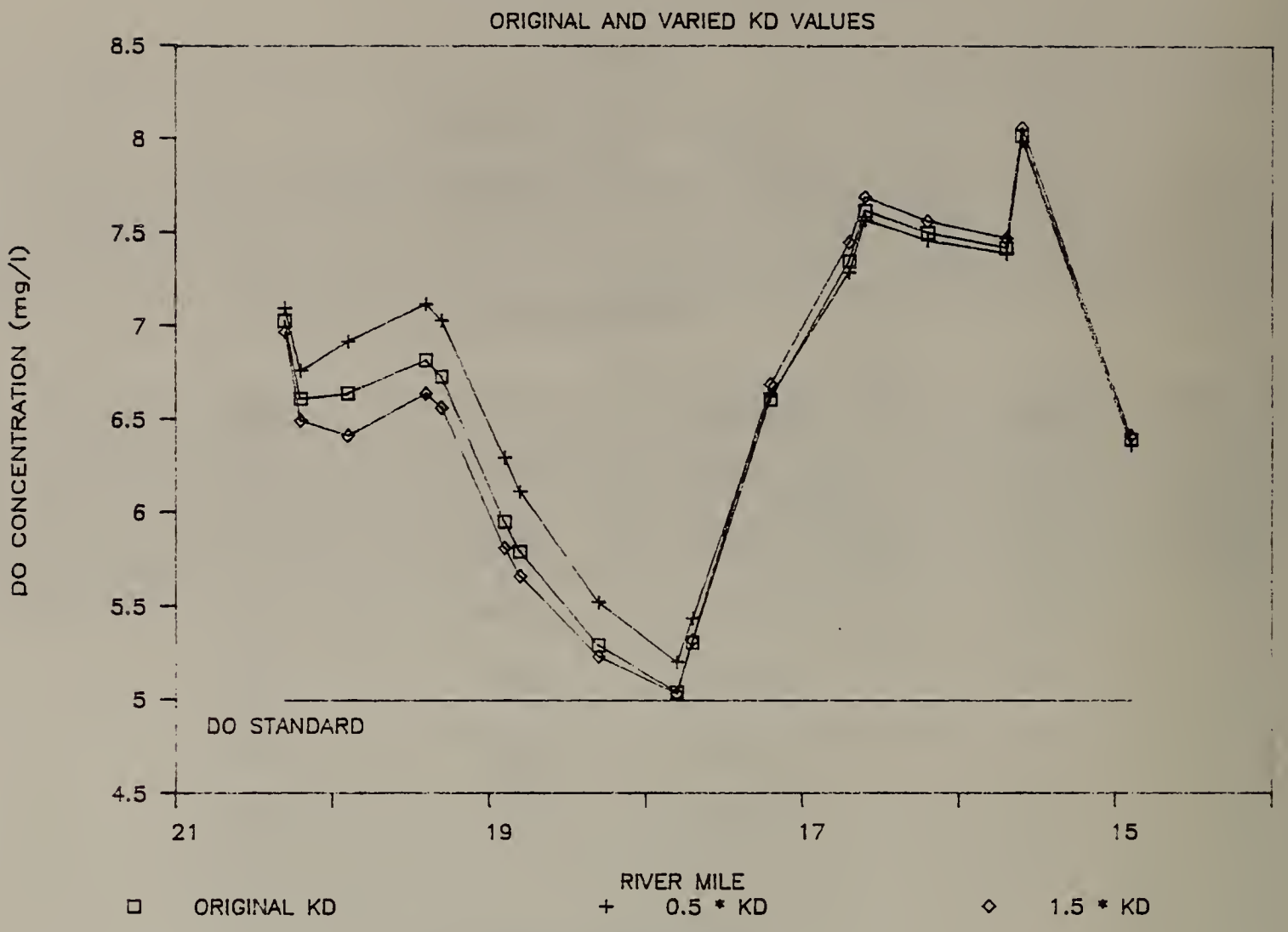


FIGURE 17

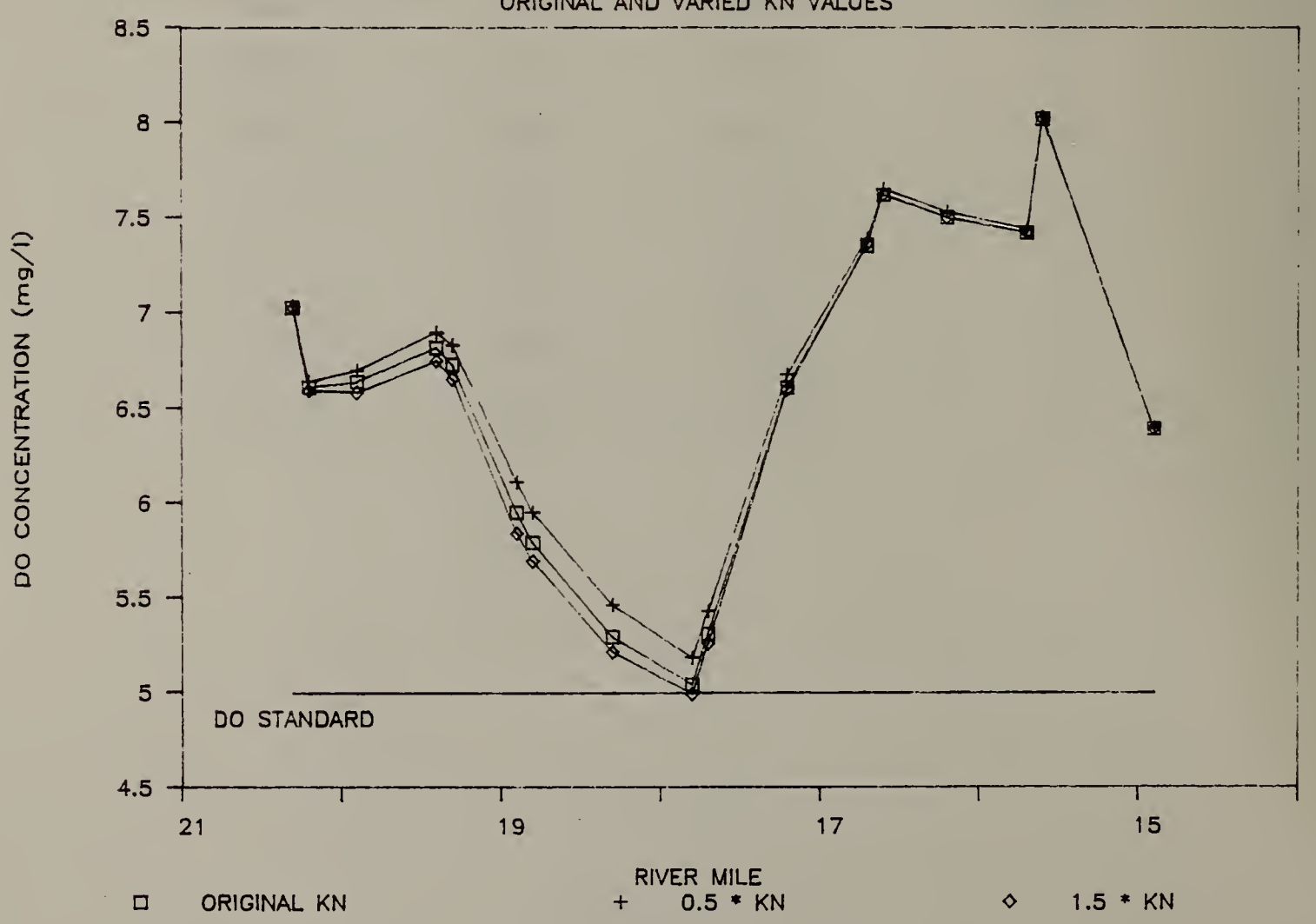


TABLE 3

K_2 SENSITIVITY ANALYSIS

REAERATION COEFFICIENTS VALUES

REACH	$0.5K_2$	LOW FLOW MODEL K_2	$1.5K_2$	2/H
1	3.6	7.2	10.8	1.825
2	0.815	1.63	2.445	0.793
3	0.605	1.21	1.815	1.003
4	1.555	3.11	4.665	1.462
5	0.08	0.16	0.24	0.388
6	0.895	1.79	2.685	4.773

SOD SENSITIVITY ANALYSIS

SOD $gm/ft^2/day$

REACH	0.5 SOD	SOD	1.5 SOD
1	0.075	0.15	0.225
2	0.125	0.25	0.375
3	0.103	0.205	0.308
4	0.025	0.05	0.075
5	0.018	0.035	0.053
6	0.325	0.650	0.975

FIGURE 18

SENSITIVITY ANALYSIS

VARIATIONS OF REAERATION COEFFICIENT K2

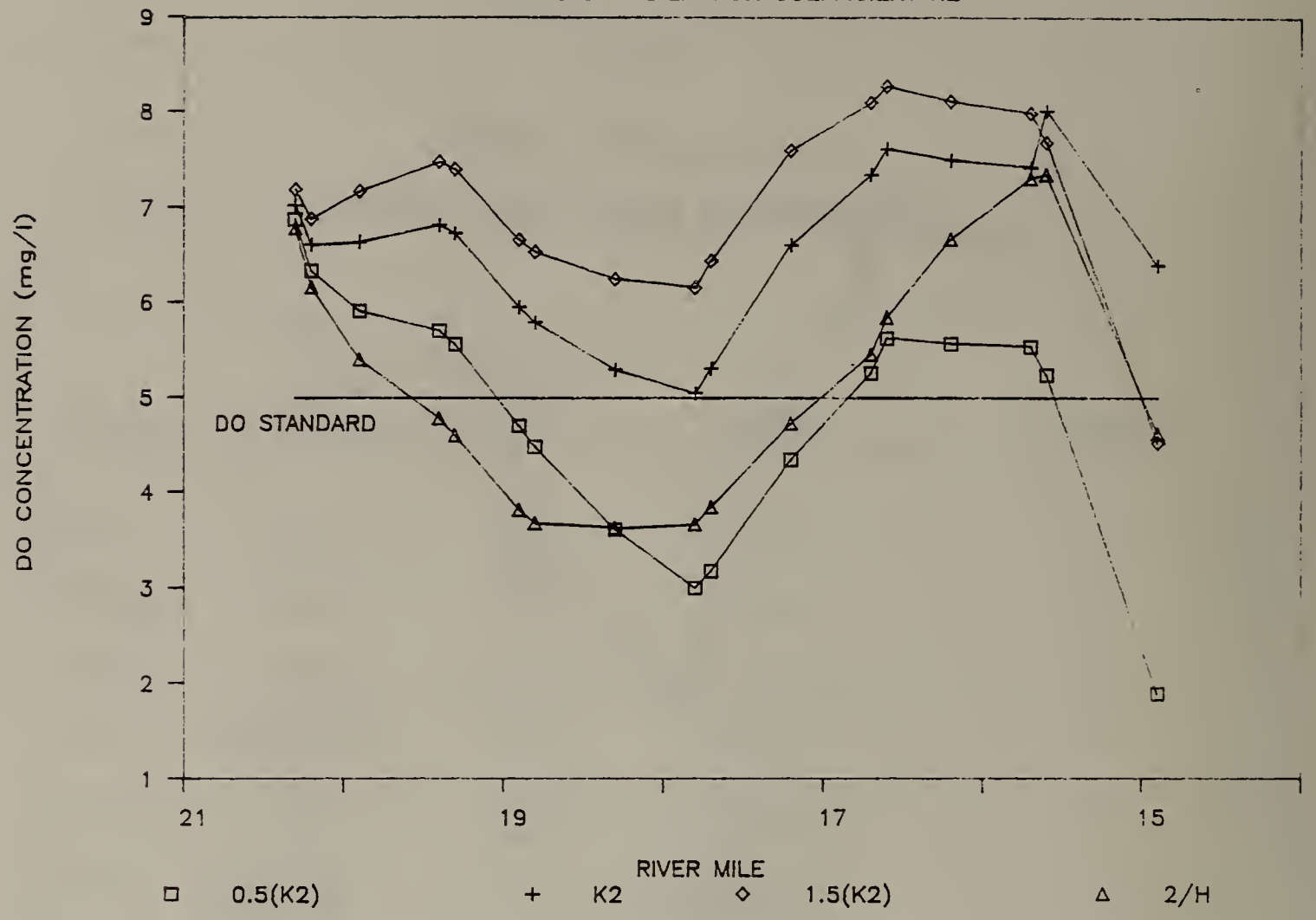
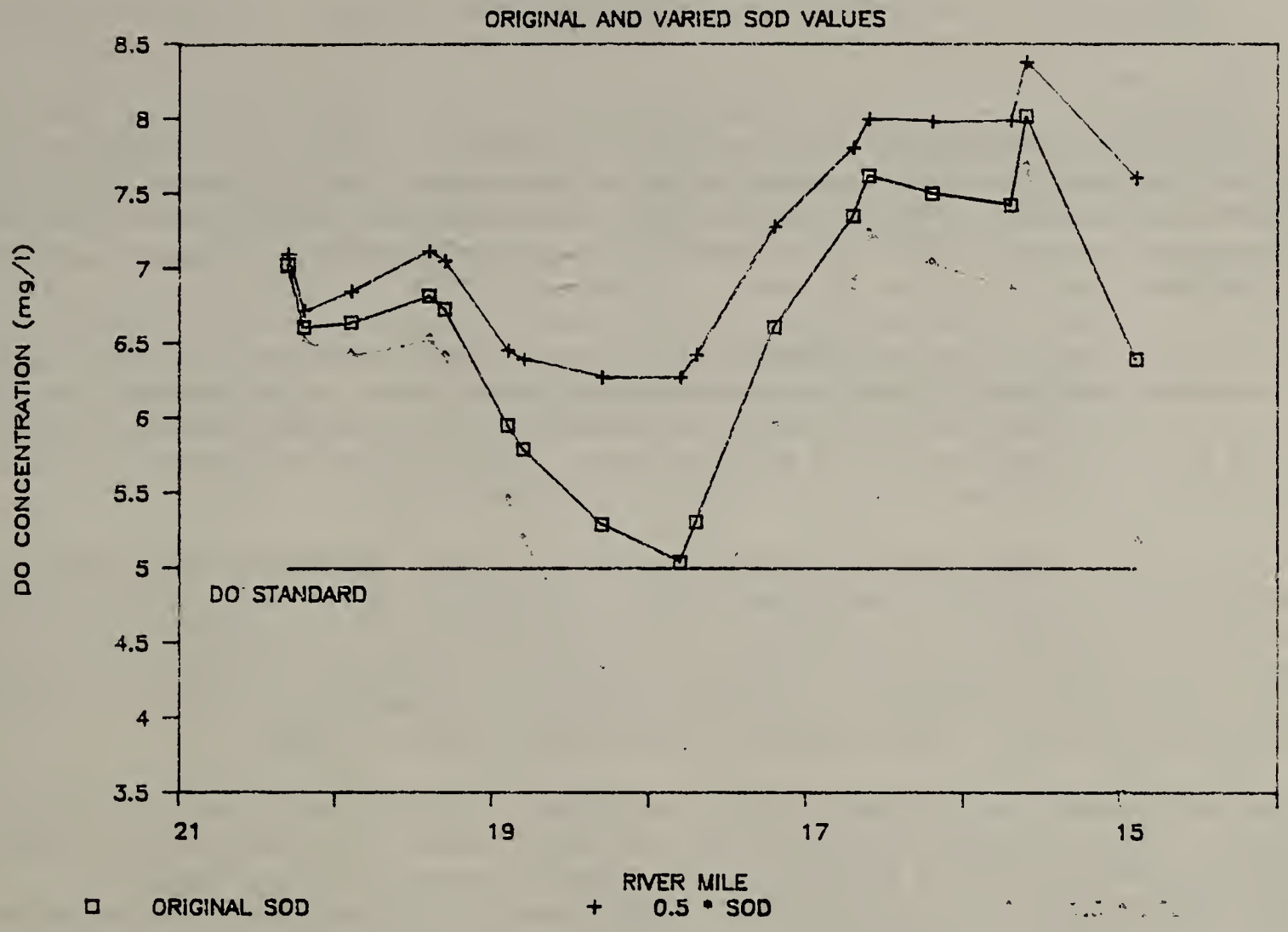


FIGURE 19

SENSITIVITY ANALYSIS



CONCLUSION

The Sturbridge Wastewater Treatment Plant (WWTP) is currently a secondary WWTP that discharges to the Quinebaug River at mile 20.2. The 0.5 MGD (0.775 cfs) plant discharges, to the Quinebaug River at a location having has a seven-day, ten-year low flow (7Q10) of 6.82 cfs, giving the wastewater effluent a 9.7:1 dilution.

The Sturbridge plant has requested an increase in flow capacity at their plant to handle wastewater from new housing developments and allow the town to remove their current sewer moratorium. Two different flow capacities were evaluated, a 0.75 MGD discharge and a 1.0 MGD discharge. With these discharges, the dilutions would be reduced to 6.9:1 and 5.6:1 respectively.

The U.S. EPA Qual2E computer model was used for the Sturbridge WWTP wasteload allocation. The model was created using the August 31, 1988 data and verified using the September 20, 1988 data. Once created low flow conditions were input and run through the model. Various BOD, NH_3 and phosphorus effluent limits were input for the 0.75 MGD and 1.0 MGD discharge. Concentrations were varied until instream dissolved oxygen concentrations were above 5.0 mg/l.

The model predicted effluent limits of 10 mg/l for BOD and 3 mg/l for NH_3 are needed for a 0.75 MGD discharge and effluent limits of 10 mg/l for BOD and 2 mg/l for NH_3 for a 1.0 MGD discharge. Both flows would also require a phosphorus limits of 1.0 mg/l and the effluent dissolved oxygen to be above 5.0 mg/l. Phosphorus concentrations were determined by varying the phosphorus concentrations in the effluent and choosing a concentration that would best keep the instream phosphorus concentration below 0.1 mg/l.

The majority of the wastewater treatment plants in Massachusetts discharging effluents to small streams with dilutions less than 10:1 require advanced limits in order to meet water quality standards. If the current plant increases its flow capacity to 0.75 MGD or 1.0 MGD the dilution would drop well below a 10:1 dilution ratio.

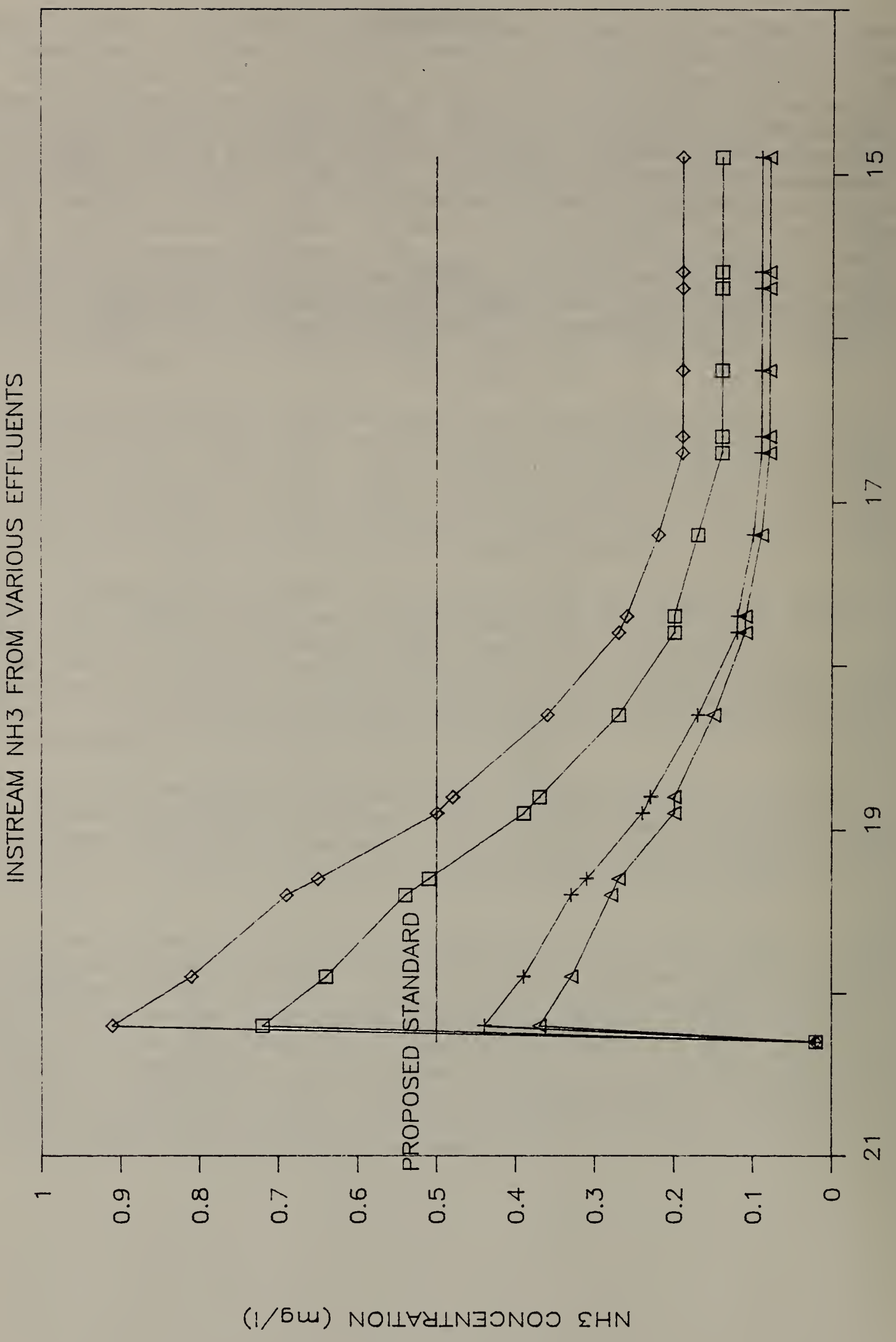
These models are deterministic but have a margin of error in their calculations because coefficients are not known with 100% accuracy and other procedures such as photosynthesis are not included. The model was created to predict average D.O. concentrations in the river. While recommended limits are calculated to have average instream dissolved oxygen concentrations to be 5.0 mg/l, any diurnal variation will cause the dissolved oxygen concentrations to drop below 5.0 mg/l. The fall of 1988 water quality surveys observed diurnal D.O. variations ranging from 0.6 mg/l to 3.2 mg/l. These concentration variations below 5.0 mg/l can help account for any error in the modeling.

When determining the required ammonia effluent concentration, the instream ammonia concentrations (in the Quinebaug below the WWTP) are considered as well as the models prediction of the effect on the instream dissolved oxygen. The Massachusetts (DWPC, 1989) Water Quality Standards have proposed a freshwater instream ammonia limit of 0.5 mg/l to protect against toxicity in the river. The various NH_3 effluent concentrations cause various instream ammonia concentrations during low flow conditions. For WWTP flows of 0.75 MGD and 1.0 MGD, the instream ammonia concentrations are plotted for effluent concentrations of 5 mg/l, 3 mg/l and 2 mg/l of NH_3 in Figure 19. Ammonia concentrations of 5 mg/l caused instream concentrations to rise above the new standards limit of 0.5 mg/l for the first mile to mile and a quarter below the discharge. The lower effluent ammonia limits never cause instream ammonia concentrations to exceed 0.5 mg/l, further confirming the recommended limits from the model.

FIGURE 20

0.75 & 1.0 MGD

INSTREAM NH3 FROM VARIOUS EFFLUENTS



RECOMMENDATIONS

Based on the predictions from the Sturbridge WWTP wasteload allocation, the following effluent limits should be required at the Sturbridge WWTP in order for water quality standards to be met during low flow conditions in the Quinebaug River:

	Average Daily	
Flow (MGD)	0.75	1.0
BOD_5 (mg/1)	10	10
SS (mg/1)	10	10
NH_3 (mg/1)	3	2
TP (mg/1)	1	1
D.O. (mg/1)	5	5

Average daily fecal coliform concentrations should not exceed 200 org/100 ml and thus some form of disinfection will be needed. If chlorine disinfection is used a chlorine residual will be established based on instream dilution.

REFERENCES

1. Camp Dresser & McKee/Resource Analysis. Wasteload Allocation Manual. June 1980.

2. Johnson, Arthur. Nashua River Wasteload Allocation Documentation. 1980

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   - French and Quinebaug Rivers 1974 Water Quality Survey Data. November 1974
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   - French and Quinebaug River Basins Water Quality Data - Wastewater Discharge Data - Biological Sampling Data 1984-1985. February 1986.
   - 1988 Upper Quinebaug River Survey - Water Quality Data - Wastewater Discharge Data - Water Quality Analysis. December 1988.

4. Tetra Tech, Inc. Rates, Constants, and Kinetic Formulations in Surface Water Quality Modeling. December 1978. EPA-600/3-78-105.

5. U.S. EPA. Computer Program Documentation for the Enhanced Stream Water Quality Model Qual2E. August 1985.

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STURBRIDGE WLA ADDENDUM #2

The Sturbridge wastewater treatment plant's wasteload allocation was performed in early 1989 and sent to EPA for review. One of the major comments was that the Quinebaug River is a cold water fishery and that instream dissolved oxygen (DO) concentrations should not dip below 6.0 mg/l. The original model was run for an instream DO concentration of 5.0 mg/l.

Using the original model, different BOD and NH3 effluent limits were run to try and obtain a minimum instream DO of 6.0 mg/l. The model, however, predicted that even with strict effluent limits of 5 mg/l BOD and 1 mg/l NH3, instream DO concentrations would not rise above 5.4 mg/l. The model was thus rerun and all the coefficients were reevaluated to check for accuracy.

First the widths and depths of the stream were compared. The widths and depths observed during the August and September surveys are listed in Table 1. The last third of this table is the widths and depths the model predicts for the low flow conditions. Some of the width and depth coefficients were changed as a result of this investigation and the new width and depth coefficients are listed in Table 2.

TABLE 2

Discharge Coefficients

Reach	a	b	α	B
1	0.129	0.2975	1.05	0.020
2	0.129	0.2975	2.50	0.004
3	0.040	0.5975	1.90	0.02
4	0.050	0.5960	0.95	0.168
5	0.02	0.0060	5.05	0.010
6	0.48	0.1780	0.125	0.557

The Kd or BOD decay rates seemed inconsistent. Usually BOD decay slowly gets smaller and smaller. In the model, reach 3 had an unusually low Kd compared to the reach above and below it. Furthermore, the BOD decay in reach 5 was listed as 0.0, which is very unlikely. The BOD decays were thus looked at and changed. The changes were run through the model with the August and September data for verification. The actual versus predicted BOD concentrations are plotted in Figures 1a and 2a. The new coefficients are listed in Table 3.

TABLE 2

Width and Depths
for the
Quinebaug River Surveys

8/31/88

Station	Width ft	Avg. Depth ft	Approx. Area sf	Approx. Vel ft/s	Approx. Flow cfs
QR01	37.3	1.73	67.5	0.34	22.4*
QR01A	64	1.19	76.16	0.31	23.5
QR01B	--	--	--	--	24.76
QR01C	112	1.4	156.8	0.17	26.06
QR01CD	61	1.53	93.33	0.28	26.06
QR01D	62	1.7	105.4	0.25	26.06
QR01E	--	--	--	--	26*
QR01F	54	0.6	32.4	0.81	26.2

9/20/88

Station	Width ft	Avg. Depth ft	Approx. Area sf	Approx. Vel. ft/s	Approx. Flow cfs
QR01	37.0	1.38	51.9	0.37	21.1*
QR01A	61.2	1.2	72.64	0.32	23.4
QR01B	--	--	--	--	24.8
QR01C	125.5	1.17	144.3	0.18	26.0
QR01CD	59.5	1.82	111.4	0.24	26.4
QR01D	62.5	1.61	101.2	0.27	26.8
QR01E	--	--	--	--	28*
QR01F	55	0.63	34.65	0.83	28.6

* Actual flow measurement

7Q10 Flow (1 MGD)

Reach	Width ft	Depth ft	Vel ft/s	Flow cfs
Above STP	27.4	1.091	0.228	6.82
1	31.476	1.096	0.243	8.37
2	14.018	2.522	0.245	8.57
3	30.059	1.984	0.145	8.67
4	35.144	1.368	0.182	8.77
5	83.867	5.161	0.02	8.77
6	29.631	0.419	0.706	8.77

TABLE 3

BOD Decay Coefficients
(All units in 1/day)

Reach	Old Kd	New Kd
1	3.080	3.080
2	3.080	2.980
3	0.534	2.214
4	2.210	1.710
5	0.000	0.300
6	1.700	0.200

To determine the affect of different SOD's, different reaeration and SOD concentrations were determined and the model was rerun. The reaeration rate for each reach was determined by using the reaeration equations of four different scientists; Dobbins and O'Connor, Owens, Thankston and Krenkel, and Langbien and Durum. Each scientist created their equations based on various stream conditions, and thus predict different reaeration rates for the same stream. The different reaeration rates are listed in Table 4 below:

Table 4
Reaeration Values for the Quinebaug

Reach	Dobbins & O'Connor	Owens	Thankston & Krenkel	Langbien & Durum
1	5.67@	7.20*	1.01	1.66
2	1.63*	1.55	0.37	0.55@
3	1.39	1.21*	0.17	0.27@
4	2.94	3.11*@	0.39	0.65
5	0.16*@	0.08	0.01	0.02
6	40.88	87.25	10.82@	17.38*

* Reaeration values used in original model
@ Reaeration values used in new model rerun

Some of the original reaeration rates were higher than other equations predicted for each reach. Different reaeration rates were then chosen according to the characteristics of each reach, and at rates that were not too much of an extreme variation from other rates. The new reaeration values used are marked by a @ in Table 4, and the original rates are marked by *. Reaches 4 and 5 were kept the same, although their corresponding SOD's were changed.

Once the new reaeration values were chosen, new corresponding SOD concentrations were determined for each reach. The new reaeration and SOD concentrations were run through the August and September 1988 survey models to determine their accuracy. The model predictions verses actual observed survey data for the August and September survey data are plotted in Figures 3a and 4a. The new reaeration and SOD values and the old values are listed in Table 5. SOD is listed as gm/ft2/day and gm/m2/day.

Table 5
K2 and SOD values by Reach

	Original Model			Revised Model		
Reach	K2 1/day	SOD g/sfd	SOD g/m2d	K2 1/day	SOD g/sfd	SOD g/m2d
1	7.2	0.150	1.63	5.67	0.08	0.87
2	1.63	0.250	2.72	0.55	0.08	0.87
3	1.21	0.205	2.23	0.27	0.015	0.16
4	3.11	0.050	0.54	3.11	0.015	0.16
5	0.16	0.035	0.38	0.16	0.040	0.44
6	17.38	0.650	7.08	10.82	0.60	6.53

The new SOD concentrations were more realistic (except reach 6). The Quinebaug River is a swift moving river where little settling will occur. The slowest sections of the river are below the treatment plant and in Westville Impoundment above the dam. The SOD values below the dam do not seem reasonable at all, but the dissolved oxygen sag does not go that far. Furthermore, the reaeration values are excessively high to compensate for the high SOD's.

The new discharge coefficients, BOD decay, reaeration and SOD rates were entered into the low flow model. Different BOD and NH3 concentrations were run for a 1.0 and 0.75 MGD discharge flows at the Sturbridge WWTP. The models predicted instream DO concentrations for each flow and pollutant loadings are presented in Figures 5a and 6a.

The model was run for a cold water fishery classification. Water temperatures were set at 68 C (20 C), and a dissolved oxygen concentration minimum limit of 6.0 mg/l was set. For a 0.75 MGD and a 1.0 MGD flow, effluent concentrations of 10 mg/l for BOD and 1.5 mg/l for NH3 would be required.

These new effluent limits are slightly stricter than the original model had predicted for a 5.0 mg/l instream dissolved oxygen. The original model, however, predicted that no effluent limit would meet the cold water fishery requirements of 6.0 mg/l. The new revised model has lower SOD concentrations which account for the lower limits.

FIGURE 1a

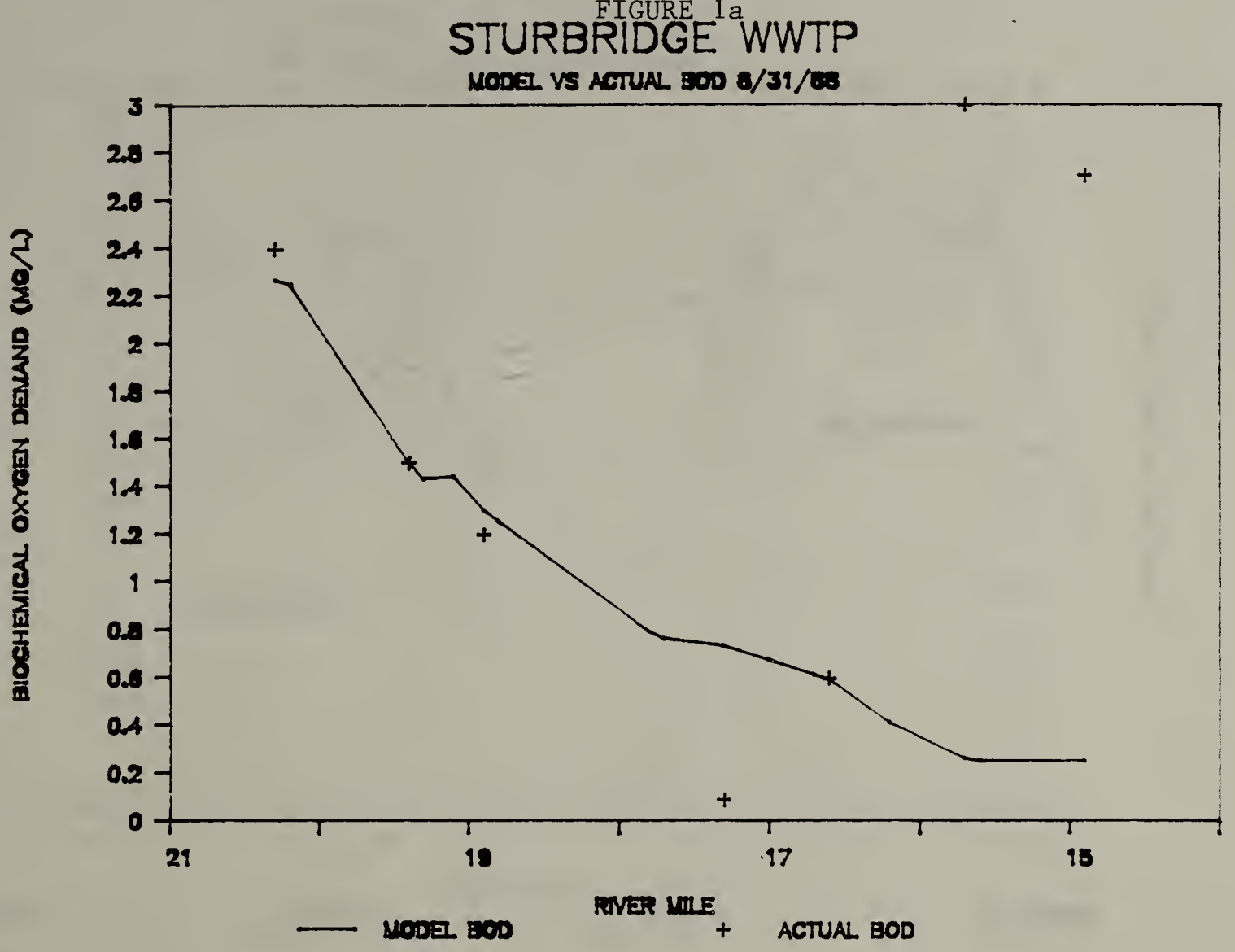

FIGURE 2a

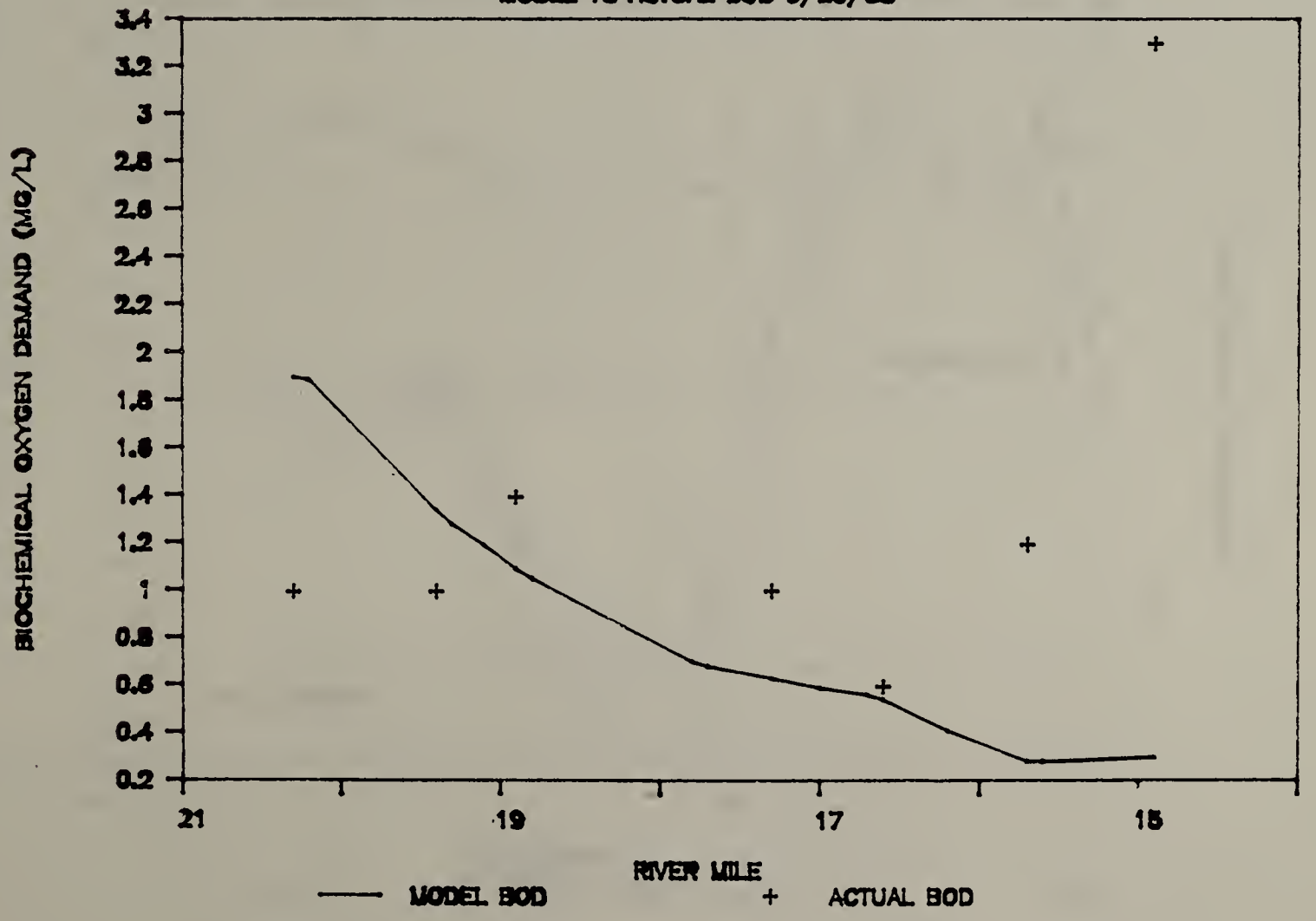

FIGURE 3a

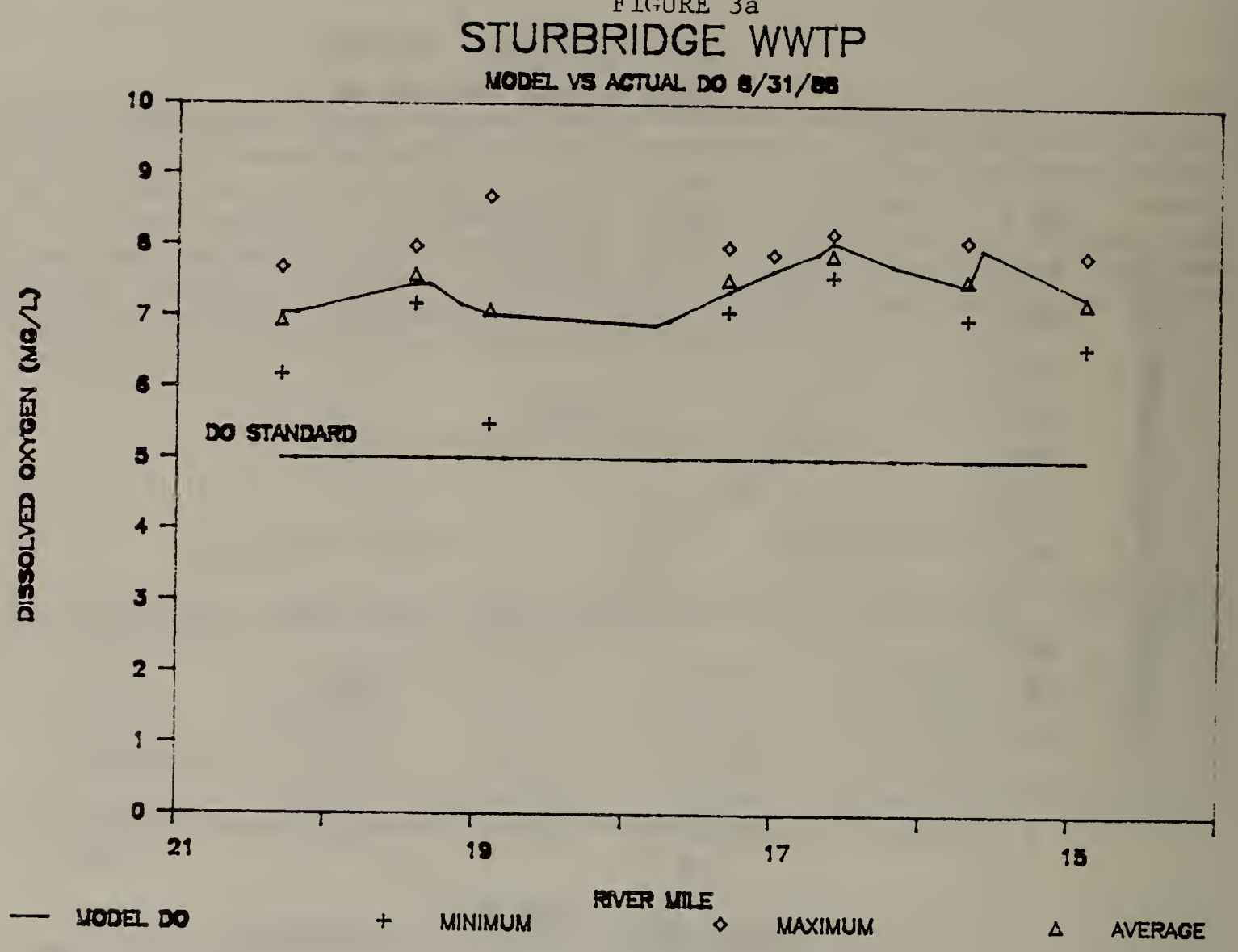

FIGURE 4a

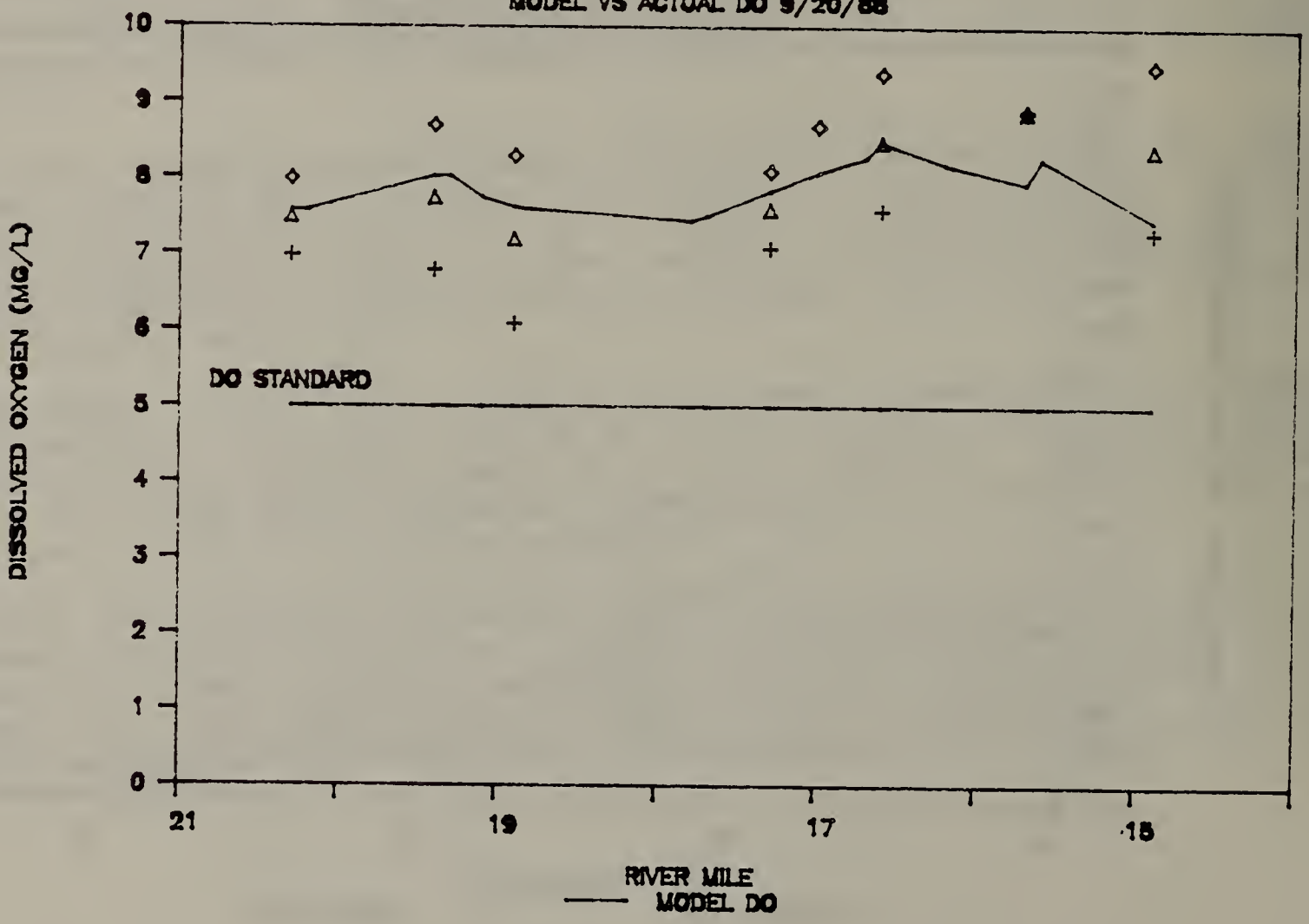

FIGURE 5a

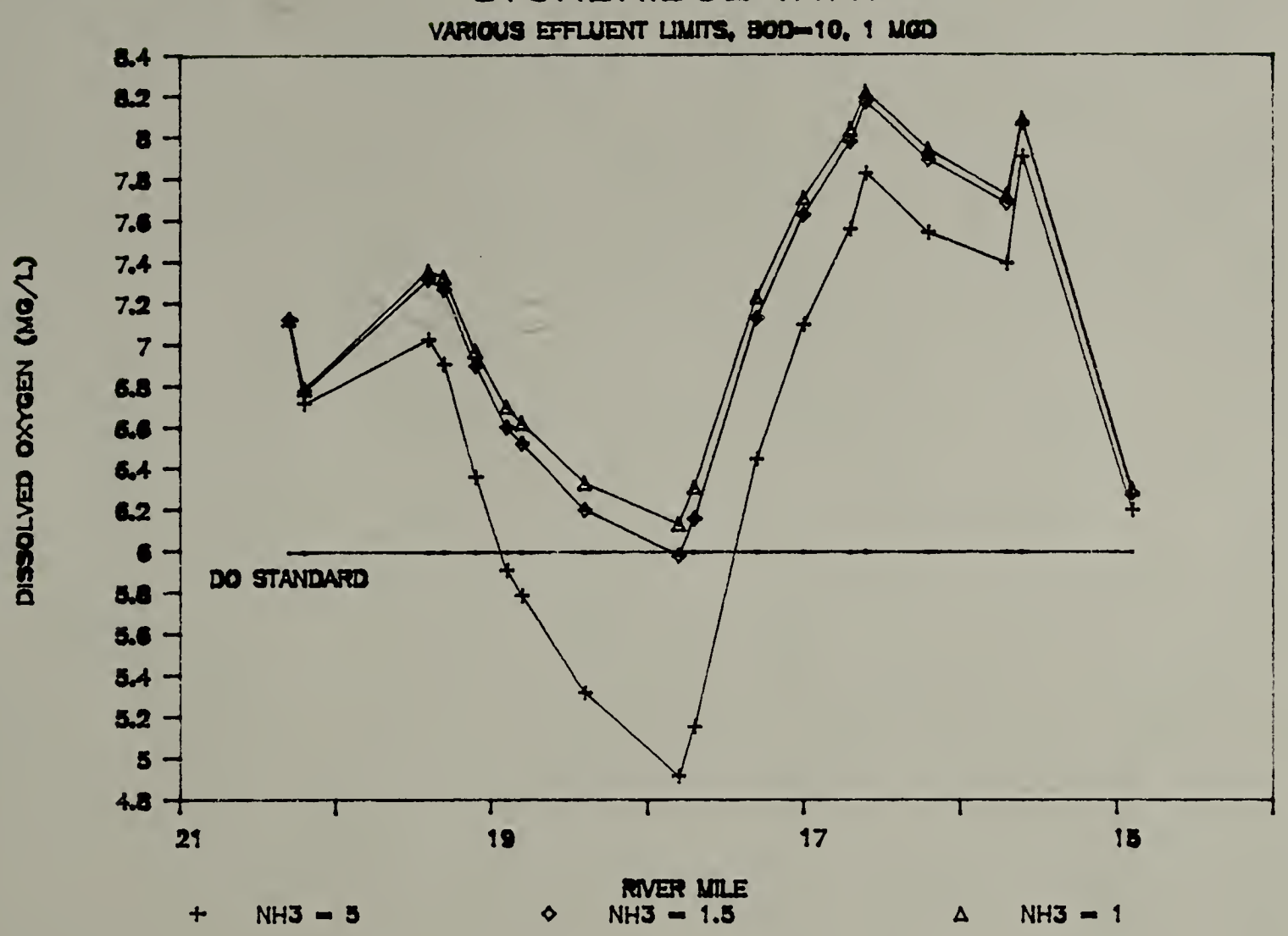
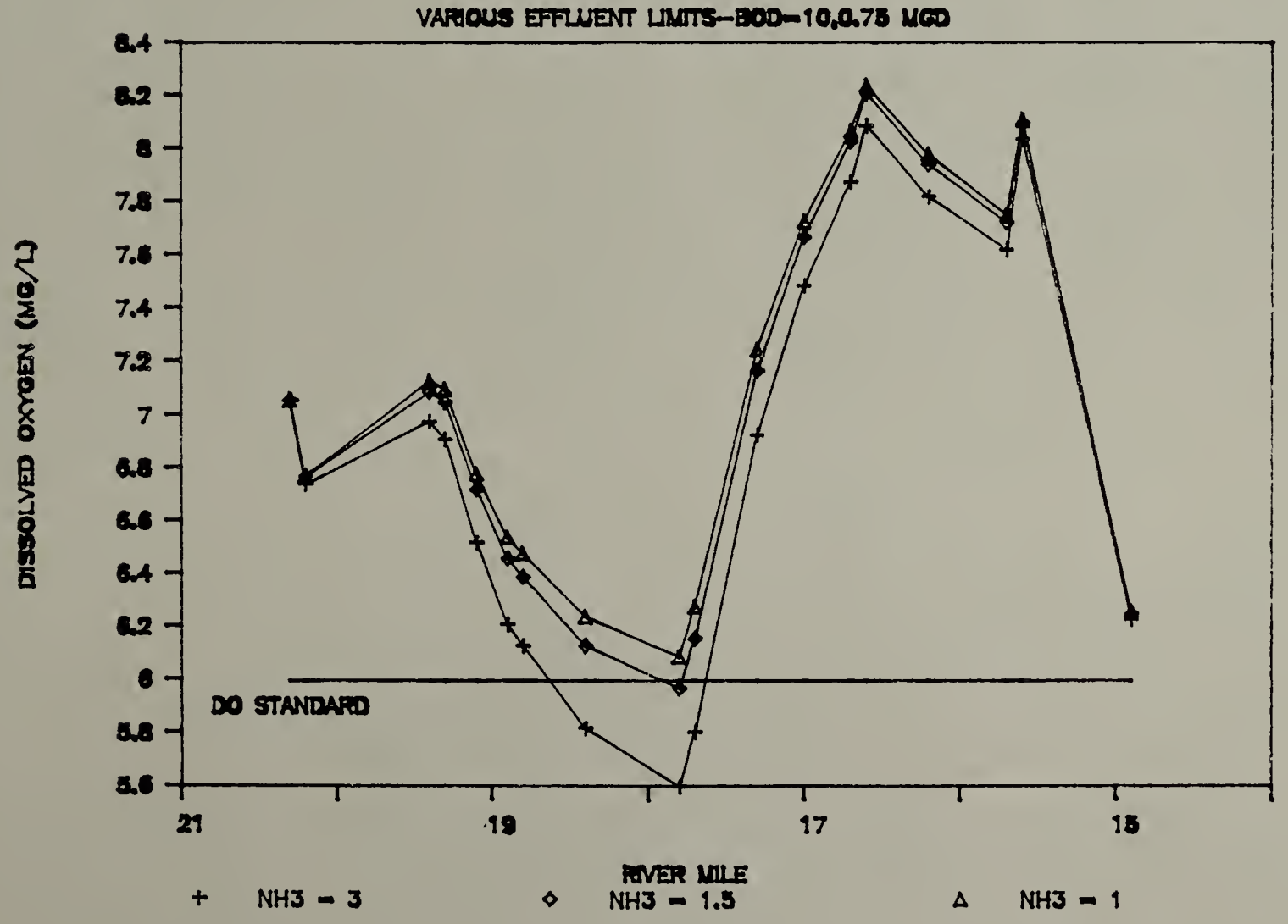
FIGURE 6a